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Technical Evaluation of Transportation Control Measures for the St. Louis Area

Peat, Marwick, Mitchell and Co.
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This report presents engineering warrants, or guidelines, to enable the East-West Gateway Coordinating Council to translate sketch planning level transportation control measures to specific projects in the St. Louis area. Warrants are documented for exclusive bus lanes on existing facilities, reversible traffic lanes, signal synchronization / modification, intersection and interchange improvements, road and bridge widening and railroad grade separation. They can be used to assist in identifying promising sites for implementing transportation control measures. In addition, analytical procedures for estimating the reduction in air pollution emissions for various transportation control measures are described. Specific procedures covered correspond to the following transportation control measures: signal synchronization, railroad grade separations, widen roads or bridges, add lanes, major new facilities, intersection and interchange improvements and simple signal turning changes.

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Technical Evaluation of Transportation Control Measures for the St. Louis Area

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I.

INTRODUCTION

The purposes of this report are to (1) provide a brief synopsis of engineering warrants, or guidelines, to enable the East-West Gateway Coordinating Council (EWGCC) to translate the sketch-planning transportation control measure (TCM) recommendations emerging from Barton-Aschman's study to specific projects, and (2) describe procedures for analyzing the impact of specific TCMs on air pollution emissions at the project level. These warrants will assist the EWGCC in identifying locations within the St. Louis region which appear suitable for specific types of TCMs. In addition, they will allow the EWGCC to determine if the characteristics of the TCMs are compatible with the actual physical, operating and usage characteristics of transportation facilities at selected locations.

Warrants for those TCMs which are of the greatest interest to the EWGCC are covered in the following two sections of this report. They include:

Improved mass transit:

- exclusive bus lanes on existing facilities;
- reversible traffic lanes for autos and other vehicles.

Traffic flow improvements:

- signal synchronization/modification;
- intersection and interchange improvements;
    - geometric improvements
    - channelization
    - adding turn lanes
    - adding signals
- road and bridge widening; and
- railroad grade separation.

The remaining sections of this report outline procedures for analyzing the following TCMs:

- signal synchronization;
- railroad grade separation;
- widen roads and bridges;
- add lanes;

- major new facilities;
- intersection and interchange improvements; and
- simple signal timing changes.

II.

IMPROVED MASS TRANSIT

Planning criteria and guidelines for implementing exclusive buslanes on existing facilities and reversible traffic lanes for autos and other vehicles are discussed in this section.

EXCLUSIVE BUSLANES ON EXISTING FACILITIES[1]

Planning Criteria

Planning and implementing exclusive buslanes on existing facilities require: (1) a high concentration of existing or planned bus services, (2) significant vehicle traffic congestion, and (3) community support for public transportation. There is little value in providing exclusive buslanes where service is poor, costly, or non-existent; where their are neither buses or congestion; and where the local community has little if any desire to maintain and improve bus service. A realistic assessment of demands, costs, benefits, and impacts is required prior to deciding to implement an exclusive buslane.

Possible objectives of plans to implement exclusive buslanes on existing facilities include:

- improving the current level of service and capacity provided (e.g., travel time, probability of obtaining a seat, etc.);
- increasing bus service reliability and safety (e.g., reducing chances of automobile-bus accidents, eliminating travel time delays attributable to automobiles, other);
- providing sufficient reserve capacity to accomodate future growth in bus patronage;
- attracting more choice automobile drivers through offering more attractive service; and
- promoting long range transit improvement programs and community development programs.

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[1] The discussion of exclusive buslanes on existing facilities is largely based on materials in the National Cooperative Highway Research Program Report 155, Bus Use of Highways Planning and Design Guidelines, Prepared by (H.S. Levenson, et. al.) Wilbur Smith and Associates (1975) and the more recent Transportation Research Circular Interim Materials on Highway Capacity, Prepared by the Transportation Research Board (1980).

There are a number of key factors which should be considered in deciding whether or not to implement an exclusive buslane. These include:

- the intensity and growth prospects of the area to be served by the exclusive buslane;
- the historic and potential future dependence of the area on bus transportation;
- characteristics of the roadway, including street width, general configuration, continuity, and overall congestion;
- current and projected bus operating speeds and reliability on the planned buslane;
- availability of alternate routes or lanes for displaced motorists so as to ensure that new problems are not created, nor existing problems transferred to other areas;
- locations of key employment centers with respect to the planned exclusive buslane;
- potential impacts on goods and service vehicle loading requirements;
- likely bus operating patterns (skip-stop, express or local);
- bus passenger loading requirements;
- community attitudes and resources;
- enforcement of buslane exclusivity and any maintenance requirements; and
- ease with which buses may enter or leave the planned exclusive buslane.

Before implementing an exclusive buslane on an existing facility, an assessment should be made of its benefits and impacts. This is important to provide a rational basis for implementation and also to assure its operational viability.

Guidelines For Implementation

Bus Streets and Bus Lanes

Exclusive buslanes may be quickly implemented, have minimal environmental impacts, and have relatively low initial costs. Exhibit II-1 presents major characteristics and capacity guidelines for bus streets and lanes. Capacities of 3,000 to 4,500 persons per hour per lane may be realized with

EXHIBIT II-1

Summary: On-Street Bus Service Options

(Bus Lanes and Bus Streets)

Item	Curb Bus Lanes-Normal Flow	Curb Bus Lanes-Contra-Flow (Extended)	Median Bus Lanes	Bus Streets
Applicability	Generally 30-60 buses/hour: 1,200-1,600 people/hour/one-way. Preferably two lanes available for other traffic in same direction. Ability to restrict turns.	40-60 buses/hour: 1,600-2,400 persons/hour/one-way. At least two lanes available for other traffic. Signal spacing greater than 500-foot intervals.	60-90 buses/hour: 2,400-3,600 persons/hour/one-way. At least two lanes available for other traffic. Ability to separate turn conflicts from buses.	60-90 buses/hour: 2,400-3,600 persons/hour/one-way. Commercially oriented frontage. Ability to service buildings (from alternate locations or in off-peak periods.) Minimum garage access requirements along street.
Principal Design Features	10' lanes delineated by paint and signs.	Lanes separated by paint or physical barrier. Opposing left turns prohibited or specially treated.	Lanes separated by paint and/or block-long pedestrian islands at least 5' wide with access at adjacent intersections.	Minimum 22-33' width. Four-lane operation on wide street.
Capacity	60-90 buses/lane/hour desirable (4,500 people) 90-120 maximum (6,000 people).	60-90 buses/lane/hour desirable (4,500 people) 90-120 maximum (6,000 people).	60-90 buses/lane/hour desirable (4,500 people) 90-120 maximum (6,000 people).	60-90 buses/lane/hour desirable For 2 lanes one-way 60 buses/lane/hour desirable.
Construction Costs Per Mile (1974 levels)	$3,000-$6,000	$4,000-$100,000	$15,000-$100,000	$500,000-$2,000,000 Costs may be lower where existing streets can be used without physical changes.
Speed Minutes/Mile	5 to 10	5 to 8	5 to 9	5 to 7
Minutes/Mile Saved	1.5 to 5.0	1.5 to 6.5	1.5 to 6.5	1.5 to 8
Remarks:	May impact curb access and deliveries. Difficult to enforce.	May impact curb access and deliveries. Self-enforcing.	Difficult for buses to leave lanes in central area. Passengers must cross traffic to board buses. Excellent potential for express service on approaches to city center.	Requires alternate traffic routes. May enhance CBD by removing cars and increasing sidewalk width. Excellent visibility. Self-enforcing. Optimal distribution in medium-sized cities.

(1 mph = 1.6 kph, 1 min/mile = 0.6 min/km)

Source: Levenson, H.S., et.al., Bus Use of Highways: Planning and Design Guidelines, National Cooperative Highway Research Report 155, 1975.

good service levels. The effectiveness of bus streets and lanes is generally limited with respect to time saving and service reliability. Primary disadvantages often include:

- limited net gains in passenger carrying capacity;
- need for suitable diversionary roadways;
- service requirements of adjacent land use;
- potential left and right turning restrictions on general traffic; and
- need to rely on high levels of enforcement.

Some general guidelines for setting and planning exclusive bus streets and lanes are:

- Bus lane and bus street installation should recognize the service needs of adjacent land uses, which often result from long-established development patterns. Deliveries should be restricted during hours that priorities are in effect or be provided from the opposite side of the street, from side streets, or (ideally) off-street. Service requirements are especially significant where contra-flow bus lanes are provided.
- Design of bus lanes should reflect available street widths and prevailing operating practices. Bus lanes should be provided whereever possible without reducing the lanes available to through traffic in the prevailing direction of flow. This may entail elimination of parking or reduction of lane width to provide additional lanes, elimination of left-turn lanes, and/or use of reversible-lane operations.
- Effective enforcement of bus lanes is essential. Bus lane proposals should be accompanied by active enforcement programs.
- Emergency vehicles - police cars, fire equipment, and ambulances - should be allowed to use bus lanes and/or bus streets. Taxis should be permitted wherever less than 60 buses use a lane in the peak hour. This will increase the utility and acceptance of bus lanes and bus streets.
- The high proportion of peak hour urban travelers using buses in downtown areas suggests increased consideration of (a) bus streets and (b) bus priorities in "auto-free zones". Bus streets can provide early action, cost effective downtown distribution for radial busways. They can also help improve the pedestrian amenity of major activity centers, particularly retail areas.

- The prohibition of curb parking, at least during the peak hours, should be prerequisite to establishing bus lanes. This measure (a) makes an additional lane available to moving traffic, thereby substantially increasing capacity; (b) reduces delays and marginal frictions resulting from parking maneuvers; and (c) allows buses easier access to stops.

- Bus routes should be restructured as necessary to make full use of priority lanes and streets. Buses may be shifted from other streets to build up patronage provided they do not exceed lane or street capacity or create passenger inconvenience. Peak hourly (one-way) bus volumes ranging from 60 to 90 will help "enforce" the bus lanes without creating excessive queuing.

- Bus priority treatments should reduce both the mean and the variance of average journey times. Because the absolute time savings may be small in many situations, emphasis should be placed on increasing service reliability. A 10 to 15 percent decrease in bus running time in the bus priority areas is a desirable objective.

- A wide application of bus lanes is necessary before schedule speeds can increase sufficiently to produce significant operating economics and/or encourage additional riding. Extended bus lanes on radial arterial streets could produce important benefits in service dependability. A saving of 1 min per mile (as by raising bus speeds from 10 to 12 miles per hour) could produce a 4-min saving if achieved over the entire length of a typical bus journey.

Contra-Flow Freeway Bus Lanes

Contra-flow bus lanes are an adoption of the reversible lane concept applied to urban freeways. They generally are characterized by low initial costs, high passenger safety and ease of enforcement.

Application of contra-flow freeway bus lanes is limited by the availability of suitable road geometry and high maintenance costs. They should only be used on multilane (six or eight lane) freeways where peak hour traffic is highly unbalanced and all automobile traffic leaves from the right. The bus lane can be separated from opposing traffic by a one lane buffer on eight lane freeways and by traffic posts on six lane freeways. Buses should run non-stop throughout the extent of the contra-flow lanes.

Normal Flow Freeway Bus Lanes

Exclusive use of existing freeway lanes by high occupancy vehicles (buses and/or carpools) pose possible safety and enforcement problems although they seem more adaptable to carpools that contra-flow lanes. Care should be exercised if an existing lane is preempted in the peak direction of travel - this practice should generally be avoided. Some general guidelines for exclusive bus lanes on freeways are as follows:

- A range of 40 to 60 buses per hour one way may be used as a warrant for exclusive use of a freeway lane;
- Major overload points on existing freeways should be identified and be used as a guide as to where special bus priority treatment may be warranted;
- It is generally not feasible to remove existing freeway lanes from auto use in the heavy flow direction and give these lanes to buses. If the freeway is already congested, reducing the lanes available to cars is likely to increase delay unless there is a significant diversion to transit; and
- Right-hand freeway lanes are not usually desirable for exclusive bus use because of frequent weaving conflicts with entering and exiting traffic.

Assessment

Most bus priority measures constitute reserve bus lanes on city streets - usually in the direction of traffic flow. However, the number of bus only streets - such as State Street in Chicago, Nicollet Mall in Minneapolis, and Chestnut Street in Philadelphia - is increasing.

Busways and reserved lanes on freeways are mainly found or are being proposed in larger American Cities - usually with a large downtown employment and heavy peak hour bus ridership. Within the last few years, there has been a tendency for medium-sized cities such as Miami and Portland to install normal flow bus and car pool lanes. The trend is to implement operationally oriented measures because of the time, costs, and complexity associated with major new construction.

Distribution of buses in central areas remains an important challenge, and communities are giving this item increased attention. Freeway-related treatments generally provide good access to the CBD perimeter, but do not substantially improve service within the downtown core. Terminals are not always located near major employment concentrations and may require secondary distribution, and curb bus lanes may not be effective. Consequently, there have been several attempts to install contra-flow bus lanes in downtown areas.

Many bus priority measures have produced important passenger benefits - especially those relating to freeways. They achieve peak hour travel time savings of 5 to 30 minutes - savings which compare favorably with those resulting from rail transit extensions.

Successful priority treatments are usually characterized by: (1) an intensively developed downtown area with limited street capacity and high all day parking costs, (2) long-term reliance on public transport, (3) highway capacity limitations on approaches to downtown (4) major water barriers which limit road access to the CBD and which channel bus flows, (5) fast non-stop bus runs for considerable distances, (6) bus piorities on approaches to or across water barriers, (7) special bus distribution within the CBD - often off-street terminals, and (8) active traffic management, maintenance, and operations programs.

Some Typical Applications

By the mid-1970's transportation planning had shifted away from costly capital intensive priority treatments that require extensive new construction to traffic management schemes that use existing facilities in a more efficient manner.

Exclusive Bus Lanes on Arterials and Commerce Business District (CBD) Streets

At least thirty cities have exclusive bus lanes on major arterials and CBD streets. The reduction in travel time has been found to depend largely on the length and number of stops along the restricted portion of the route and the level of enforcement. Example applications include:

- Having started a preferential street program for buses in 1974, San Francisco has implemented many exclusive transit lanes since that time (e.g., Sutter St. (1.02 miles), Post St. (0.74 miles), Mission St. (0.65 miles), others). Some were exclusively used by buses only during the peak travel periods while others remained exclusive all day.

- In 1976 Washington, D.C. implemented serveral reserve bus lanes on arterials. Reserve curb lanes for both express and local buses on major arterials are available to suburban commuters entering the CBD during the morning peak travel period and in the opposite direction during the evening period. During off-peak periods these lanes are used primarily for parking.

- St. Louis implemented reserve bus lanes on Olive and Lindall Streets in 1976. A curb lane is reserved for buses running between Kingshighway Boulevard and the downtown CBD during the peak travel periods. In addition, a curb lane is reserved for buses on Locust Street between 4th and 12th Streets in the downtown area between 6:00 AM and 6:00 PM.

Normal or Concurrent-Flow Freeway High Occupancy Vehicle (HOV) Lanes

In recent years there have been a number of attempts to implement reserved lanes on freeways (e.g., Santa Monica Freeway in Los Angeles, I-95 in Miami, the Southeast Expressway in Boston, and the Banfield Freeway in Portland, Oregon). These projects have met with differing degrees of success and failure. The two projects that entailed taking a lane from general traffic and re-dedicating it to HOV's (the Southeast Expressway and the Santa Monica Freeway) ended amid much controversy and public dispute. The remaining two projects which required the creation of a new lane have enjoyed varying degrees of success. Problems encountered included underutilization of the reserve lane, high violation rates, and enforcement difficulties. Some general observations are:

- Re-dedication of an existing lane for concurrent-flow priority use is not recommended;
- Concurrent- flow lanes on freeways should be separated from other lanes by closely spaced plastic inserts to prevent weaving and by a concrete barrier or a safety lane to minimize problems of accidents or violations; and
- A left shoulder is desirable for vehicles in distress and necessary for enforcement.

Contra -Flow Freeway Lanes

These are a number of successful contra-flow freeway lanes for buses that have been operating for several years. These include U.S. 101 in Marine County, California; I-495 approaching New York City's Lincoln Tunnel; and the Long Island Expressway approaching New York City's Queens-Midtown Tunnel.

REVERSIBLE TRAFFIC LANES FOR AUTOMOBILES AND OTHER VEHICLES[1/]

On roadways where directional flow is unbalanced -- e.g., a 65-35 directional split -- reversible lanes may be used during the peak period to expand the capacity of the roadway in the peak flow direction. Although, like contra-flow freeway lanes traffic in reversible lanes flows against the usual direction, reversible lanes usually imply roadways not separated by a median strip. Therefore, reversible lanes are an inexpensive and effective technique for increasing the operating efficiency of existing facilities since they add peak-period directional capacity without high capital expenditures.

[1/] This discussion of reversible traffic lanes is largely based on materials in Kenyani, B.I. and Putnam, E.S., Transportation System Management: State-of-the-Art, Prepared by Interplan Corporation for the U.S. Department of Transportation, 1977 and Seeburger, S., Transportation System Management Institutional and Planning Research, Working Paper No. 11, Traffic Engineering Improvements, Prepared by A. M. Voorhees Associates for the U.S. Department of Transportation and the U.S. Environmental Protection Agency, April 1979.

Applicability

On most urban arterials, the volume of inbound traffic is typically much greater during the a.m. peak period and much less during the p.m. peak period relative to outbound traffic. This situation results in underutilization of the street in the light traffic direction and congestion in the other. By implementing a reversible lane, that is assigning underutilized lanes in the light volume direction to serve the peak traffic flow, overall roadway capacity and level of service may be enhanced.

On an undivided arterial street, of three or more lanes, the following conditions may warrant implementing a reversible lane:

- Sixty-five percent or more of the traffic moves in one direction during the peak travel periods;
- The remaining roadway in the off-peak direction is adequate;
- There is continuity in the route and width of the street; and
- There is no median, and left turns and parking can be restricted. For example, on three and five lane roads, the center lane may operate as a two-way left-turn lane during the off-peak, but may operate as a one-way lane in the direction of the heaviest flow during the peak travel periods.

Experience suggests that the best traffic flow in the peak flow direction may be realized when reversible lanes are combined with progressive signal timing.

Experience With Reversible Lanes

Atlanta, GA, Arlington, VA, Los Angeles, CA, Chicago, IL, Milwaukee, WI and Washington, D.C. are among the cities in which reversible lanes have been successfully implemented. Specific experiences of two cities, Atlanta and Arlington, are described below:

- Atlanta uses reversible lanes on several roadways in the city. A 3.5 mile three-lane section of Memorial Drive was converted from a traffic configuration of two outbound lanes and one inbound lane to one lane in each direction and a reversible center lane. Peak period travel time on the converted section was reduced by about 25 percent in the heavy flow direction while in the light flow direction there was an insignificant change in travel time;

- Arlington, VA installed a 2.8 mile reversible lane on Wilson Boulevard, a four lane road where, during peak periods, reversible lane concepts and bus only reserve lane concepts are combined. Bus travel times have been reduced between two and three minutes on the 2.8 mile section. Inbound morning motorists use the two center lanes and buses use the inbound curb lane. During evening peak hours, the system operates in a similar fashion in the reverse direction.

Cost Impacts of Reversible Lanes

Signing and lane use control devices, usually overhead lane signals, and pavement striping comprise the basic costs for reversible lanes. Depending on the length of the system, maintenance and enforcement personnel may also be needed.

Assessment

Reversible traffic lanes have been successful in improving traffic safety, reducing peak period traffic congestion, reducing travel times and increasing speeds. There are relatively few disadvantages. Under certain circumstances travel speeds may decrease in the non-peak direction; however, the speed increases in the peak flow direction are likely to benefit a larger volume of traffic than that impacted by the speed reductions in the non-peak direction. Another problem may occur when only one lane is available for off-peak direction flow. Under such conditions, if a vehicle becomes disabled or must turn, long queues may form behind that vehicle.

III.

TRAFFIC FLOW IMPROVEMENT WARRANTS

Planning guidelines for implementing signal synchronization, intersection and interchange improvements, road and bridge widening and railroad grade separations are discussed in the following section.

SIGNAL SYNCHRONIZATION AND MODIFICATION[1]

The traffic signal cycle length -- the time required for the display of all signal indications -- and the control concept for the traffic network or route are the major elements influencing the number of vehicles that can pass through an intersection. By determining the most efficient use of signal time on an intersection-by-intersection basis, the amount of time available for individual movements at a group of intersections may be optimized and vehicular throughput increased.

Signal Timing Optimization

The key objectives of optimizing signal timing at an intersection are the minimization of delay and congestion, and the increase in safety for all road users. The standard practice in traffic engineering is to deal with the safety aspect by assigning a clearance interval (yellow or yellow plus all red) which is judged to be safe and which does not cause increased delays. There exists no explicit rule (that is one supported by traffic engineering studies) for calculating an optimum clearance. Once an adequate clearance interval is determined, the principal concern in timing optimization becomes determination of the amount of green time to assign to each approach to the intersection in such a way as to minimize delays and congestion, and thus maximize the flow of traffic through the intersection. There are a number of computation techniques, such as the TRANSYT program, which can be used to optimize signal tuning.[2]

The degree of traffic flow improvement produced by signal timing optimization will generally depend on one or more of the following factors:[3]

- The level of quality of the preexisting signal timing plan;
- The level of quality or skill used in determining the new traffic signal timing plan;

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[1] This discussion is based on materials in Kenyani, B.I. and Putnam, E.S., Transportation System Management: State-of-the-Art, prepared by Interplan Corporation for the U.S. Department of Transportation, 1977 and Basu, S., Transportation System Management Institutional and Planning Research, Working Paper No. 12, Traffic Signal Improvements, prepared by A.M. Voochees.

[2] Robertson, D.I. and Gower, P., User Guide to TRANSYT Version 6, Transport and Road Research Laboratory, Crowthorne, Berkshire, England, 1977.

[3] Wagner-McGee Associates, Overview of the Impacts and Costs of Traffic Control System Improvements, prepared for the Federal Highway Administration, 1980.

- The extent of constraints on the timing plan imposed by the design features, configuration, and traffic characteristics of the intersection, roadway or network being optimized; and

- The quality and completeness of the traffic data being used to develop the new timing plan.

Coordinated or Synchronized Traffic Control Systems

To achieve smooth flow on a street or a network, it is not sufficient to ensure that each point of local conflict (usually intersections) are efficiently controlled. Unless the controls at each of these locations are coordinated (e.g., through progressive interconnection, through a computerized traffic-responsive control system) continuous smooth flow on the street system is unlikely to be achieved. It is particularly important that signals in close proximity to each other (spacing of one-half mile or less) be coordinated, to prevent inefficient traffic flow.1/ The most efficient traffic signal synchronization can generally be achieved through implementation of computerized traffic-responsive control systems.

Traffic-actuated signals, tied to computerized signal systems, have been installed in or are planned for more than 100 cities. Computerized traffic control systems are suitable primarily in cities with populations over 80,000.

INTERSECTION AND INTERCHANGE IMPROVEMENTS

Warrants and guidelines for considering the implementation of the following types of intersection and interchange improvements are discussed below:

- geometric improvements;
- channelization;
- adding turn lanes; and
- adding traffic signals.

Intersection performance is a critical determinant of the effective use of highways and streets. It is the focal point of conflicts and congestion since it is common to two or more roadways. As the number and severity of intersection conflicts increase, there is an increased need for improved traffic control. Improved traffic control may be used to increase intersection capacity, reduce and prevent accidents and to create and protect major streets.

1/ Ibid.

Geometric Improvements[1]

Most traffic engineering improvements are low cost options. Altering facility geometrics may cost up to $1 million or more and therefore, are typically considered only when lower cost options prove ineffectual. Geometric improvements include the following types of actions:

- Widen intersection approaches;
- Align offset approaches to an intersection; and
- Grade separation.

Widening Approaches - Widening intersection approaches will generally increase throughput by increasing capacity. About a 25 to 60 percent improvement in service volume may be expected per 10 foot widening of approach width. The extra width may be increased by prohibiting parking, using available median width, using existing unused right-of-way, acquisition of additional right-of-way, or any combination of these.

Align Offset Approaches (Jog Correction) to an Intersection - Offset approaches at intersections cause complicated and time-consuming traffic operations and delays. Such intersections may require a minimum of three signal phases to operate safely when signalized. This often restricts capacity of the intersection. Considerations for correcting jog intersections include:

- High incidence of accidents;
- Significant delay which would be eliminated by improved geometrics;
- Signal retiming, channelization and lane restrictions have not been effective in improving operations; and
- Cost of reconstruction.

Grade Separation - Grade separation of an intersection is generally warranted where total traffic volume is high, left turn volumes are heavy, and other intersection improvements have not proven effective. This improvement is the most costly of those mentioned, costs usually ranging to $1 million or more. Grade separation can have the highest impact on the surrounding environment and must consequently be given careful study. Grade separation often causes congestion problems to appear further up or downstream from the improved intersection.

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[1] This discussion is based on material S. Seeburger, Transportation System Management Institutional and Planning Research, Working Paper No. 11, Traffic Engineering Improvements, prepared by A.M. Voorhees and Associates for the Urban Mass Transportation Administration, the Federal Highway Administration, and the U.S. Environmental Protection Agency, April, 1979.

Channelization[1/]

Channelization techniques may be used to optimize the capacity of an intersection. Channelization - directing traffic into defined paths on roadways - can reduce or eliminate potential hazards to motorists and pedestrians and minimize confusion in traffic flow. Generally, channelization requires using islands, pavement markings, or other means of facilitating traffic flow to separate potential areas of conflict, improve safety, provide for the maximum utilization of available road space, and regulate traffic movement and speed. Effective channelization may improve speeds by as much as 10 mph. When turning volumes are heavy, specially designated turn lanes may smooth vehicular flow.

When turning movements conflict with and impede through traffic, turn restrictions, a common form of channelization, may provide a solution. Preventing right turns may minimize vehicle/pedestrian conflicts, whereas preventing left turns minimizes vehicle/vehicle conflicts. Indications are that part-time turn restrictions may be effective during peak periods; however, compliance is reportedly better when turn restrictions are in effect all of the time. The following conditions may warrant establishing turn restrictions:

- The left turn volume exceeds 20 percent of the total approach volume;
- Left turns constitute 10 percent of the total movement on a given street;
- Left turn movements interfere with straight-through movements of 15,000 vehicles per day, regardless of the number of lanes;
- A left turn or right turn movement interferes with pedestrian crosswalk volumes in excess of 2,000 persons per hour;
- 600 vehicles conflict with 1,000 or more pedestrians per hour;
- Seven vehicles turn per green interval for several successive signal changes;
- More than three intersection accidents that involve turning vehicles occur within a 12-month period;
- The number of traffic lanes available at the intersection will accommodate only a single movement in each direction, and there is an appreciable demand for left turns; and
- Periodic gaps in opposing traffic do not occur due to traffic signals ahead.

The Redwood City and Sunnyville highway optimization experiments with left turn prohibitions observed that:

- When left turns are prohibited at all intersections along a major street within the core area, left-turning traffic can be successfully diverted if

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1/ This discussion is largely based on material in Kenyani, B.I. and Putnam, E.S., Transportation System Management: State-of-the-Art, prepared by Interplan Corporation for the U.S. Deparment of Transportation, 1977.

a convenient alternate route is available. Traffic flow on the major streets is improved because of reduced volume and elimination of turning delays.

- When no alternate route is available, left-turning traffic must turn right to circle the block. Traffic flow is improved only if pedestrian interference with right-turning vehicles does not increase turning delay. Traffic volume on the major street is usually not reduced.
- Drivers cannot be expected to observe the continuous left turn prohibition when an alternate route is not convenient unless such a prohibition is strictly enforced.

In those situations where channelization techniques are applicable, the Transportation and Traffic Engineering Handbook (published by the Institute of Transportation Engineers) suggests the following general rules be followed:

- Islands should be arranged so that the driving path seems natural and convenient;
- There should only be one path for the same intersectional movement;
- A few well-placed, large islands are better than a confusion of small islands;
- Islands should be offset 2 feet or more from the edge of the roadway;
- Adequate approach-end treatment is required to warn drivers and to permit gradual changes in path and speed;
- Curving roadways should have radii and width adequate for the governing design vehicle; and
- Adequate visibility should be provided to drivers approaching the intersection. There should be no hidden obstructions and islands should be well defined. In many cases, some form of illumination will be needed, depending on the type of curb used to outline the island.

Adding Turn Lanes[1/]

An exclusive turn lane, left or right, is only warranted when there are heavy turning movements, since it may significantly reduce capacity if it is not fully utilized. Ideally, separate turn lanes should be provided by increasing the intersection's approach width of the pavement, thereby preventing interference with through movements. This type of intersection modification is often not possible for urban intersections. During peak hours it is often necessary to allow turns from more than one lane because of high flows. This is particularly true for one-way streets and access routes to and from freeways.

[1/] Discussion based on material from Pignataro, L.J., Traffic Engineering Theory and Practice, Prentice-Hall, Inc., Englewood Cliffs, New Jersey, 1973.

There are no commonly accepted warrants for implementing a separate left turn lane. Factors that should be considered prior to making a decision include:

- Accident experience involving left-turning vehicles with opposing traffic, sideswipe collisions, and rear end collisions:
- Volumes of left-turning traffic and opposing traffic;
- Delay to left-turning and straight-through movements;
- Physical layout of the intersection; and
- Cost of construction.

Adding Traffic Signals[1]

Traffic signals are used to alternately assign the use of the intersection to one stream of traffic, then to the other. Right-of-way is allocated by time separation of conflicts.

Since a traffic signal systematically assigns the right-of-way to competing movements, it will improse a delay on all approaches to the intersection. Therefore, it is critical to be certain that a traffic signal is justified before installing one. The Manual On Uniform Traffic Control Devices (MUCTD) details eight key warrants, at least one of which should be met before a traffic signal is installed.[2] These warrants relate to:

- minimum vehicular volume;
- interruption of continuous traffic;
- minimum pedestrian volume;
- school crossing;
- progressive movement;
- accident experience;
- systems; and
- combination of warrants.

Each of these are briefly discussed in Exhibit III-1. More details may be found in pertinent sections of the MUCTD which have been furnished in Appendix A at the back of this memorandum.

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[1] This discussion is primarily based on material from Pignataro, L.J., Traffic Engineering Theory and Practice, Prentice-Hall, Inc., Englewood Cliffs, New Jersey, 1973.

[2] U.S. Department of Transportation, Federal Highway Administration, Manual On Uniform Traffic Control Devices, 1978.

EXHIBIT III-1

SUMMARY DESCRIPTIONS
OF
TRAFFIC SIGNAL WARRANTS

- Minimum Vehicular Volume. This warrant is intended to be applied where the volume of intersecting traffic is the primary reason for consideration of traffic signal control.

- Interruption of Continuous Traffic. This warrant applies to operating conditions where the traffic volume on a major road or street is so heavy that cross traffic on the minor street suffers excessive delay or danger in entering onto, or crossing, the major street.

- Minimum Pedestrian Volume. This warrant is based on a combination of vehicle and pedestrian volumes which are considered hazardous at a given intersection. Signals installed at isolated intersections under this warrant should be of the pedestrian-actuated type.

- School Crossing. A traffic signal may be warranted at an established school crossing if a study shows the gaps in the vehicular traffic, in terms of frequency and size, are not sufficient to safely provide for street crossings by school children. In particular, if gaps of sufficient duration to allow for crossings without interruption of traffic flow occur on the average of less than one per minute, children may become impatient and endanger themselves by attempting to cross the street during an inadequate gap. Under such conditions, some form of traffic control is needed to create adequate gaps in the traffic stream.

- Progressive Movement. This warrant is more directly concerned with a signal-controlled intersection as part of a system, rather than as an isolated element. For consistency of approach, however, it seems reasonable to discuss all signal warrants at the same time. Where signals are widely separated, it may be impossible to maintain coordinated control, since vehicle platoons spread out over long distances. Under such circumstances, it may be justifiable to place an intermediate signal at a location where it would otherwise be unwarranted to maintain proper coordination of platoon movements.

EXHIBIT III-1
(Continued)

- Accident Experience. This warrant comes into play when an adequate trial of less restrictive measures, with satisfactory enforcement and observance, has failed to reduce the accident frequency. Under these conditions, the occurence of a number of accidents involving personal injury or property damage in a twelve-month period may justify the installation of a signal.

- Systems. As with street progressions, signal installations under this warrant are designed to encourage concentration and organization of traffic. Where the progressive warrant was concerned with vehicular movement along a street or arterial, the systems warrant is oriented toward benefiting network flow.

- Combination of Warrants. In some cases, no single MUTCD warrant may be totally satisfied. If, however, a condition exists where any combination of the first three warrants is satisfied to within 80 percent of specified values, a signal may be justified. It is recommended, however, that other remedial measures, which incur less delay to traffic, be tried prior to installation of signals.

Source: Pignataro, L.T., Traffic Engineering Theory and Practice, Prentice Hall, Inc., Englewood Cliffs, New Jersey, 1973.

ROAD AND BRIDGE WIDENING

Narrow lanes and intermittent hazardous obstructions near the edge of the roadway increase driver tension and frequently cause drivers to select larger than normal headways and/or lower speeds. This tends to lower the capacity of narrow bridges and lanes. Maximum roadway capacities are achieved when lanes are at least 12 feet in width. The reduction in capacity and level of service associated with widths below this standard may be found for different highway facilities in the Highway Capacity Manual.

A particularly important application of this strategy is in situations where the road suddenly becomes more narrow over a short section due to the need to pass over a bridge or because of a narrow highway segment. If the level of service and capacity on these segments are severely impacted by the less than ideal width, then the possibility of widening these sections should be considered.

RAILROAD GRADE SEPARATIONS[1]

The maximum improvement to a grade crossing is separation of the grades of the railroad and highway. Although this alternative entails a large expenditure of funds, the benefits that result -- including the reduction in highway congestion and delay, the value of time saved by vehicle operators, contribution to energy conservation, improvements to the environment (particularly the reduction in air pollution emissions), increased safety as well as improvements in railroad operations -- may justify these expenditures. Grade separation should be considered specifically in improvements to railroad facilities and in the design or reconstruction of highway routes. Rails used for high speed railroad passenger services should have no at grade crossings.

Identification and Prioritization of Railroad Grade Crossing Projects

Potential projects involving grade crossing improvements are generally identified and prioritized based on accident data or safety factors, citizens' complaints, vehicle traffic volumes and time delays, existing grade crossing characteristics and various combinations of these factors which are often summarized by a priority or hazard index.

Although it is desirable to provide optimum treatment at grade crossings, funding for such capital expenditures is rarely available. Available funds are usually allocated to those locations and investments which are most likely to achieve the safety and economic benefits. Most organizations use some type of priority ranking or hazard index formula to indicate which crossings have the greatest need for improvement. These techniques are generally interpreted in a flexible way by experienced and qualifed professional engineers and regulatory or administrative agencies. Exhibit III-2 presents eleven of the most popular hazard index formulas.[2] Tests of these formulas on various grade crossings

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[1] This discussion is largely based on material from the Railroad-Highway Grade Crossing Handbook Technology Sharing Report, U.S. Department of Transportation, Federal Highway Administration, August, 1978.

[2] Other areas have more complicated priority or hazard indices which require use of a computer to handle the large amount of data needed.

EXHIBIT III-2

SELECTED HAZARD INDEX FORMULAS

Peabody and Dimmick Formula

$$A_5 = 1.28 \frac{V^{0.170} \times T^{0.151}}{P_f^{0.171}} + K$$

Mississippi Formula

$$H.I. = \frac{\frac{SDR}{8} + A_5}{2}$$

New Hampshire Formula

$$H.I. = VTP_f$$

The Ohio Method

$$H.I. = A_f + B_f + G_f + L_f + N_f + SDR$$

Wisconsin Method

$$H.I. = \frac{T\left(\frac{V}{20} + \frac{P^1}{50}\right)}{5} + SDR + A_e$$

Contra Costa County Method

$$H.I. = TZ\left(1 - 2.718^{\frac{-Vt}{1000Z}}\right)$$

The Oregon Method

$$H.I. = [V_1T_1P_f + 1.4V_2T_2P_f]\frac{A_e}{A_5}$$

North Dakota Rating System

$$H.I. = [N_f + L_f] + [P_f + D_f + G_f + X_f] + (VT_f) + SDR$$

Idaho Formula

$$H.I. = V_f \times T_f(CB_f + SDR + N_f + Y_f)$$

Utah Formula

$$H.I. = \frac{T}{1000}\left[\left(\frac{P}{10} + \frac{F}{20} + \frac{S}{30}\right) + SDR + N_f + X_f + R_f\right] + 2A_e + \frac{P^1}{100{,}000}\left(\frac{P}{10} + \frac{F}{20} + \frac{S}{30}\right) \quad P_f$$

City of Detroit Formula

$$H.I. = \frac{T}{1000}\left[\left(\frac{P}{10} + \frac{F}{20} + \frac{S}{30}\right) SDR + N_f + X_f + R_f\right](100\% - \%P_f) + 2A_e$$

SYMBOLS

A_5 = Expected number of accidents in 5 years.
A_e = Accident experience.
A_f = Accident probability factor.
B_f = Train speed factor.
CB_f = Type and speed of train factor.
D_f = Alignment of track and highway factor.
F = Number of freight trains in 24 hours.
G_f = Approach gradient factor.
H.I. = Hazard index.
K = Additional parameter.
L_f = Angle of crossing factor.
N_f = Number of tracks factor.
P = Number of passenger trains in 24 hours.
P^1 = Number of pedestrians in 24 hours.
P_c = Protection coefficient.
P_f = Protection factor.
R_f = Road approach factor.
S = Number of switch trains in 24 hours.
SDR = Sight distance rating.
t = Time crossing is blocked.
T = Average 24-hour train volume.
T_1 = Average daylight train volume.
T_2 = Average train volume during dark hours.
T_f = Train volume factor.
V = Average 24-hour traffic volume.
V_1 = Average daylight traffic volume.
V_2 = Average traffic volume during dark hours.
V_f = Traffic volume factor.
VT_f = Exposure factor.
X_f = Condition of crossing factor.
Y_f = Severity factor.
Z = Number of traffic lanes.

Source: Richards, H.A. and Bridges, G.S., Traffic Control and Roadway Elements - Their Relationship to Highway Safety: Chapter 1 - Railroad Grade Crossings, Automative Safety Foundation, Washington, D.C., 1968.

Note: Values of P_f for use with New Hampshire formula

Automatic Gates	**= 0.1**
Flashing Lights	**= 0.6**
Signs only	**= 1.0**

resulted in similar rankings for each of them. Three variables or factors common to all these formulas are: (1) the relative effectiveness of various types of control devices, (2) the probability of conflict between trains and vehicles, and (3) sight distance ratings. Values for relative hazards with specific traffic control devices are shown in Exhibit III-3. More recent data, based on a 1974 study by the California Public Utilities Commission, is shown in Exhibit III-4.

The National Railroad-Highway Grade Crossing Inventory provides sufficient data to estimate three of the hazard indices shown in Exhibit III-2. With the addition of accident and sight data, all of the hazard indices may be computed using data from the National Railroad-Highway Grade Crossing Inventory.

Improvement Based on Warrants

The preceeding discussion outlines techniques for ranking a group of railroad grade crossings according to their priority of need. The determination of whether to grade separate the crossing or take another specific action cannot readily be answered without conducting a benefit/cost analysis of alternate options. The decision to install a grade separation or select another option should be made on the basis of the expected reduction in accidents and casualties and the initial capital costs plus annual operating costs. Relative effectiveness of the various options is shown in Exhibits III-3 and III-4. Therefore, by comparing accident and casuality reductions and other impacts associated with each of the options, determination can be made on choice of the appropriate option and on priority for crossing improvement in terms of an investment decision. It should be noted that Pennsylvania has developed a detailed set of procedures for developing hazard indices and benefit/cost ratios.[1] Despite the use of hazard indices and benefit/cost ratios, professional engineering judgement is still required.

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[1] Railroad-Highway Grade Crossing Potential Hazard Index, Pennsylvania Department of Transportation, Harrisburg, Pennsylvania, 1976.

EXHIBIT III-3

THE RELATIVE HAZARD RELATIONSHIPS FOR TRAFFIC CONTROL DEVICES AT RAILROAD GRADE CROSSINGS

Type of Device	Relative Hazard
Crossbucks *	1.00
Stop Signs	0.58
Wigwags**	0.34
Flashing Lights	0.20
Automatic Gates	0.11

Source: Factors Influencing Safety at Highway-Rail Grade Crossings, National Cooperative Highway Research Program, Report 50, 1968.

*Crossbucks are two pieces of wood at 90 degree angles which say "RR Crossing." They are mounted on a pole,

**Wigwags are the same as crossbucks except the two pieces of wood are mounted on a pendulum. They are not used very frequently,

EXHIBIT III-4

RELATIVE EFFECTIVENESS OF TRAFFIC CONTROL DEVICES

Device	Relative Accident Occurrence
Crossbucks	1.00
Wigwags	0.63
Flashing Lights	0.33
Automatic Gates	0.13

Device	Fatalities per Accident		Injuries per Accident	
	Rural	Urban	Rural	Urban
Crossbucks	0.32	0.13	0.82	0.55
Wigwags	0.17	0.08	0.56	0.46
Flashing Lights	0.19	0.10	0.42	0.45
Automatic Gates	0.09	0.04	0.27	0.28

Source: "The Effectiveness of Automatic Protection In Reducing Accident Frequency And Severity At Public Grade Crossings In California". California Public Utilities Commission. 1974.

IV.

PROCEDURES FOR ANALYZING TRANSPORTATION CONTROL MEASURES SIGNAL SYNCHRONIZATION

In order to estimate air pollution emissions without detailed analysis of all locations planning to synchronize or coordinate traffic signals, the air pollution emissions associated with traffic flow improvements brought about by improved synchronization of signals may be analyzed as follows:

A. FACILITY SCALE ANALYSIS

1. It is assumed that Average Daily Traffic (ADT) counts are available for the facility, or facility segment, to be analyzed. This data may be found on highway inventory worksheets or other data sources if available. It is also assumed that most congestion, if not all, occurs during the peak travel hours and that during a typical weekday there are 4 peak travel hours and 20 non-peak travel hours. Although the analysis approach presented may be applied during both travel periods, it is likely that the most significant air pollution emission impacts will occur during the peak travel period.

2. In order to conduct the analysis for the peak and non-peak travel periods, it is necessary to obtain the fraction of ADT which occurs during the peak travel hour. If such information is not readily available, it may be estimated using a factor from Table 1. In general, the analysis must be carried out separately for each direction. Therefore, a Directional Factor (DF) which indicates the fraction of traffic in each direction must be applied to the peak hour travel volume. Directional factors may be obtained from highway inventory sheets or, if such information is not available, from "typical" combined directional and peak hour factors as shown in Table 1 for the appropriate type of facility (freeway, expressway, etc.).

$$
\begin{aligned}
\text{DPHV}_1 &= (\text{ADT}) \times \begin{pmatrix}\text{fraction of ADT}\\ \text{in peak hour}\end{pmatrix} \times \text{DF}_1 \\
&= \text{ADT} \times \text{CDF}_1
\end{aligned}
$$

where:

DPHV_1 = directional peak hour volume in direction 1

ADT = average daily traffic

DF_1 = directional factor in direction 1

CDF_1 = combined directional/peak hour factor

3. Preliminary air pollution emission impacts of traffic signal synchronization may be estimated as follows:

A. Obtain current speed estimates (prior to signal synchronization) using capacity and speed parameters for specific facility and area types from UROAD link speed and capacity tables modified for the St. Louis area as shown in Table 2.

Facility Type Codes:

1. freeways
2. expressways
3. arterials (2 way with parking)
4. arterial (1 way)
5. centroid (local and collector streets aggregated)
6. arterial (2 way without parking)

Area Type:

1. Central Business District (CBD)
2. Fringe
3. Residential
4. Outer Central Business District
5. Rural

Table 2 yields the mean free (noncongested) speed and the possible capacity per lane for a specific highway facility and area type. Current speeds may then be estimated as:

$$S_1 = \frac{So}{1 + 0.15\,(v/_c)^4} = \frac{So}{1 + 0.15\,(DPHV_{1}/_{C})^4}$$

where:

S_1 = average vehicle speed in direction 1

So = mean free speed from Table 2

C = "practical capacity" = 0.75 x ("possible" capacity) x
(per lane from Table 2)

(no. of lanes in one direction)

B. Rather than attempt to evaluate specific traffic signal system improvements, an exercise that would require considerable data and the use of either computer procedures or extremely time consuming manual procedures, the resulting improvement in average travel speed may be estimated from Figure 1. This exhibit is based on actual data on traffic signal system improvements in several urban areas. The extent of speed improvement will depend upon "the before condition" of the traffic signal system. For non-interconnected signals speeds have been observed to increase between 16 to 25 percent after installing advanced computer based signal controls.

C. The reduction in air pollution emissions as a result of implementing traffic signal system improvements may largely be captured through examining changes in average travel speed along the road, or road segment, where improvements are to be made. Formulae for estimating the net change in vehicle emissions during the peak travel periods are as follows:

Peak travel period emissions prior to improved traffic signal systems $= \sum_1 (4) \times (DPHV_1) \times d \times e(S_1)$

Peak travel period emissions after traffic signal system improved $= \sum_1 (4) \times (DPHV_1) \times d \times e(S_1')$

Net reduction in peak period travel emissions $= \sum_1 4 \times d \times (DPHV_1) \times ((e(S_1) - e(S_1'))$

where:

d = length in miles of road, or road section, over which traffic signals have been coordinated or interconnected,

$e(S_1)$ = emissions per vehicle-mile at speed S_1 prior to traffic signal system improvements.

$e(S_1')$ = emissions per vehicle-mile at speed S_1' after traffic signal system improvements.

S_1 = average vehicle speed on road, or road segment, in direction 1 prior to traffic signal improvements, obtained in Step 3.A from Table 2.

S_1' = average vehicle speed on road, or road segments, in direction 1 after traffic signal system improvements. Generally of the form, $S_1' = S(1+K)$ where K is the fractional improvement in speed as determined from Figure 1.

Some Caveats:

- The TCM analysis procedure relies heavily on past experience of other urban areas where traffic conditions and signal system improvements may not be the same as that characterizing St. Louis;
- It has been assumed that the air quality impacts on roads intersecting the road on which traffic signal system improvements have been made are relatively negligible.

B. AREAWIDE ANALYSES

This procedure is designed to analyze an extensive, areawide signal synchronization program which is impractical to define at the intersection or facility level. In cases where the facility can be defined and worked with, it is better to use the more exact procedure discussed in A.

1. Identify major arterials and calculate total ADT on these roads and their length (d). (Do not include freeways or expressways. Do include major routes leading to freeways and expressways.)
2. Assume traffic is uniformly distributed on these roads.
3. Calculate Directional Peak Hour Volume ($DPHV_1$) from ADT using CDF from Table 1.

   $$DPHV_1 = ADT \times CDF$$

4. Assume that % signals improved equals % traffic affected. Calculate affected traffic (% signals improved x DPHV).
5. Estimate average speed for areas(s).
6. Determine type of change in signalization and use Figure 1 to determine effect of change on speed. Calculate new speed (S').

Emissions reduction during peak travel period.

Before $(4) \times (DPHV_1) \times d \times e\,(S)$

After $(4) \times (DPHV_1) \times d \times e\,(S')$

d = length of road

e(S) = emissions per vehicle-mile at speed S (prior to traffic signal system improvements)

e(S') = emissions per vehicle-mile at speed S' (after traffic signal system improvements)

S = average speed on roads before traffic signal improvements

S' = average speed on roads after traffic signal improvements

TABLE 1

PERCENTAGE OF AADT IN PEAK HOUR FOR ONE DIRECTION AND BOTH DIRECTIONS BY PEAK HOUR, 30TH HIGHEST HOUR, AND 200TH HIGHEST HOUR, BY TYPE OF FACILITY

TYPE OF FACILITY	PERCENTAGE OF AADT IN PEAK HOURS					
	ONE DIRECTION			BOTH DIRECTIONS		
	PEAK HOUR	30TH HIGHEST HOUR	200TH HIGHEST HOUR	PEAK HOUR	30TH HIGHEST HOUR	200TH HIGHEST HOUR
Rural:						
Freeway	23.6	15.4	11.4	18.3	13.5	10.9
Expressway	21.5	14.1	10.6	19.2	12.7	9.7
Highway with more than 2 lanes	21.2	13.7	10.3	16.4	12.7	9.9
2-Lane two-way highway	—	—	—	19.7	13.6	11.2
Urban:						
Freeway	15.0	12.7	10.7	13.6	11.0	9.6
Expressway	14.6	11.4	8.9	11.6	9.5	8.3
Street with more than 2 lanes	13.8	11.1	9.6	12.0	10.0	8.7
2-Lane two-way street	—	—	—	13.4	10.6	9.0

Source: 1965 Highway Capacity Manual, Table 3.12, p. 40.

TABLE 2

FLORISSANT 5-ITERATION ASSIGNMENT USING BASENET.CCI8

80CT80 21.40.21 URCAD REPCRT 10

LINK SPEED AND CAPACITY TABLES

AREA TYPE 1

		FACILITY TYPE					
LANES		1	2	3	4	5	6
1	CAPACITY	1750.	800.	600.	700.	10000.	600
	SPEED	45.0	35.0	20.0	22.0	10.0	22.0
2	CAPACITY	1750.	800.	600.	700.	10000.	600.
	SPEED	45.0	35.0	20.0	22.0	10.0	22.0
3	CAPACITY	1750.	800.	600.	700.	10000.	600.
	SPEED	45.0	35.0	20.0	22.0	10.0	22.0
4	CAPACITY	1750.	800.	600.	700.	10000.	600.
	SPEED	45.0	35.0	20.0	22.0	10.0	22.0
5	CAPACITY	1750.	800.	600.	700.	10000.	600.
	SPEED	45.0	35.0	20.0	22.0	10.0	22.0
6	CAPACITY	1750.	800.	600.	700.	10000.	600.
	SPEED	45.0	35.0	20.0	22.0	10.0	22.0
7	CAPACITY	1750.	800.	600.	700.	10000.	600.
	SPEED	45.0	35.0	20.0	22.0	10.0	22.0
8	CAPACITY	1750.	800.	600.	700.	10000.	600.
	SPEED	45.0	35.0	20.0	22.0	10.0	22.0
9	CAPACITY	1750.	800.	600.	700.	10000.	600.
	SPEED	45.0	35.0	20.0	22.0	10.0	22.0

AREA TYPE 2

		FACILITY TYPE					
LANES		1	2	3	4	5	6
1	CAPACITY	1750.	1000.	550.	550.	10000.	800.
	SPEED	45.0	40.0	25.0	29.0	15.0	25.0
2	CAPACITY	1750.	1000.	550.	550.	10000.	800.
	SPEED	45.0	40.0	25.0	29.0	15.0	25.0
3	CAPACITY	1750.	1000.	550.	550.	10000.	800.
	SPEED	45.0	40.0	25.0	29.0	15.0	25.0
4	CAPACITY	1750.	1000.	550.	550.	10000.	800.
	SPEED	45.0	40.0	25.0	29.0	15.0	25.0
5	CAPACITY	1750.	1000.	550.	550.	10000.	800.
	SPEED	45.0	40.0	25.0	29.0	15.0	25.0
6	CAPACITY	1750.	1000.	550.	550.	10000.	800.
	SPEED	45.0	40.0	25.0	29.0	15.0	25.0
7	CAPACITY	1750.	1000.	550.	550.	10000.	800.
	SPEED	45.0	40.0	25.0	29.0	15.0	25.0
8	CAPACITY	1750.	1000.	550.	550.	10000.	800.
	SPEED	45.0	40.0	25.0	29.0	15.0	25.0
9	CAPACITY	1750.	1000.	550.	550.	10000.	800.
	SPEED	45.0	40.0	25.0	29.0	15.0	25.0

UNITSO CAPACITY IN %VEH/LANE /HR
SPEED IN MI/HR

TABLE 2
(continued)

AREA TYPE 3

LANES		FACILITY TYPE 1	2	3	4	5	6
1	CAPACITY	1750.	1100.	550.	900.	10000.	800.
	SPEED	50.0	40.0	30.0	32.0	15.0	28.0
2	CAPACITY	1750.	1100.	550.	900.	10000.	800.
	SPEED	50.0	40.0	30.0	32.0	15.0	28.0
3	CAPACITY	1750.	1100.	550.	900.	10000.	800.
	SPEED	50.0	40.0	30.0	32.0	15.0	28.0
4	CAPACITY	1750.	1100.	550.	900.	10000.	800.
	SPEED	50.0	40.0	30.0	32.0	15.0	28.0
5	CAPACITY	1750.	1100.	550.	900.	10000.	800.
	SPEED	50.0	40.0	30.0	32.0	15.0	28.0
6	CAPACITY	1750.	1100.	550.	900.	10000.	800.
	SPEED	50.0	40.0	30.0	32.0	15.0	28.0
7	CAPACITY	1750.	1100.	550.	900.	10000.	800.
	SPEED	50.0	40.0	30.0	32.0	15.0	28.0
8	CAPACITY	1750.	1100.	550.	900.	10000.	800.
	SPEED	50.0	40.0	30.0	32.0	15.0	28.0
9	CAPACITY	1750.	1100.	550.	900.	10000.	800.
	SPEED	50.0	40.0	30.0	32.0	15.0	28.0

AREA TYPE 4

LANES		FACILITY TYPE 1	2	3	4	5	6
1	CAPACITY	1750.	1000.	550.	650.	10000.	800.
	SPEED	45.0	35.0	20.0	24.0	15.0	22.0
2	CAPACITY	1750.	1000.	550.	650.	10000.	800.
	SPEED	45.0	35.0	20.0	24.0	15.0	22.0
3	CAPACITY	1750.	1000.	550.	650.	10000.	800.
	SPEED	45.0	35.0	20.0	24.0	15.0	22.0
4	CAPACITY	1750.	1000.	550.	650.	10000.	800.
	SPEED	45.0	35.0	20.0	24.0	15.0	22.0
5	CAPACITY	1750.	1000.	550.	650.	10000.	800.
	SPEED	45.0	35.0	20.0	24.0	15.0	22.0
6	CAPACITY	1750.	1000.	550.	650.	10000.	800.
	SPEED	45.0	35.0	20.0	24.0	15.0	22.0
7	CAPACITY	1750.	1000.	550.	650.	10000.	800.
	SPEED	45.0	35.0	20.0	24.0	15.0	22.0
8	CAPACITY	1750.	1000.	550.	650.	10000.	800.
	SPEED	45.0	35.0	20.0	24.0	15.0	22.0
9	CAPACITY	1750.	1000.	550.	650.	10000.	800.
	SPEED	45.0	35.0	20.0	24.0	15.0	22.0

UNITS: CAPACITY IN #VEH/LANE /HR
SPEED IN MI/HR

TABLE 2
(continued)

AREA TYPE 5

		FACILITY TYPE					
LANES		1	2	3	4	5	6
1	CAPACITY	1750.	1100.	550.	900.	10000.	800.
	SPEED	55.0	45.0	30.0	32.0	15.0	28.0
2	CAPACITY	1750.	1100.	550.	900.	10000.	800.
	SPEED	55.0	45.0	30.0	32.0	15.0	28.0
3	CAPACITY	1750.	1100.	550.	900.	10000.	800.
	SPEED	55.0	45.0	30.0	32.0	15.0	28.0
4	CAPACITY	1750.	1100.	550.	900.	10000.	800.
	SPEED	55.0	45.0	30.0	32.0	15.0	28.0
5	CAPACITY	1750.	1100.	550.	900.	10000.	800.
	SPEED	55.0	45.0	30.0	32.0	15.0	28.0
6	CAPACITY	1750.	1100.	550.	900.	10000.	800.
	SPEED	55.0	45.0	30.0	32.0	15.0	28.0
7	CAPACITY	1750.	1100.	550.	900.	10000.	800.
	SPEED	55.0	45.0	30.0	32.0	15.0	28.0
8	CAPACITY	1750.	1100.	550.	900.	10000.	800.
	SPEED	55.0	45.0	30.0	32.0	15.0	28.0
9	CAPACITY	1750.	1100.	550.	900.	10000.	800.
	SPEED	55.0	45.0	30.0	32.0	15.0	28.0

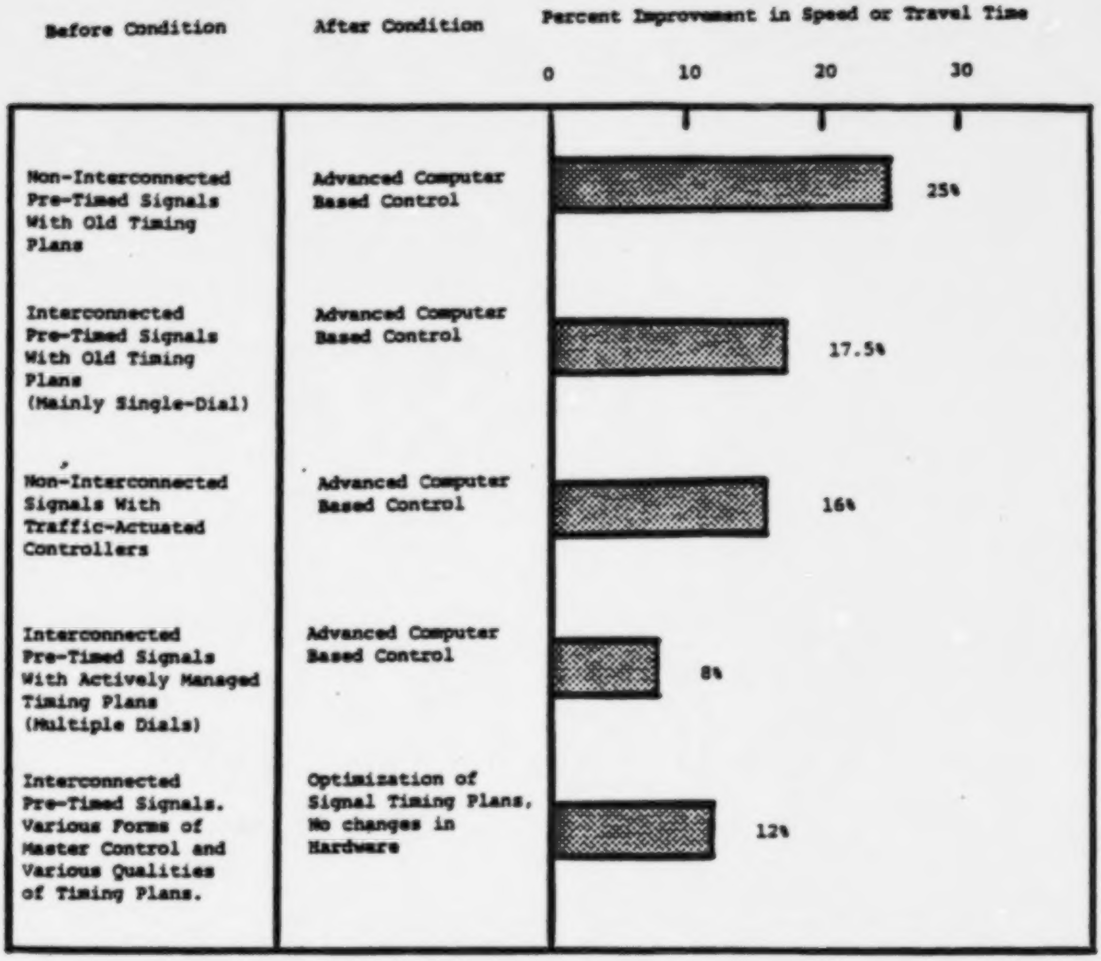
```
UNITSO CAPACITY IN ZVEH/LANE /HR
       SPEED IN MI/HR

UROAD LLL) Z INFORMATION 0 188K BYTES OF CORE NEEDED FOR NETWORK INPUT

ROA2 2600 Z INFORMATION 0  OF ON OLDHR AT NODE 4040  858 LINKS READ
```

FIGURE 1

COMPARATIVE IMPACTS OF TRAFFIC SIGNAL SYSTEM IMPROVEMENTS



Source: Wagner-McGee Associates, Inc., Overview of the Impacts and Costs of Traffic Control System Improvements, prepared for the Federal Highway Administration, March 1980, p. 20.

V.

PROCEDURES FOR ANALYZING TRANSPORTATION CONTROL MEASURES PROVIDE RAILROAD GRADE SEPARATIONS

Most at-grade railroad-highway crossings occur over single lane roads or two lane highways. In urban areas there are relatively more at-grade crossings on two lane highways than in rural areas. In general, providing a grade separation in the form of a bridge or other facility will eliminate air pollution emissions associated with delays experienced by vehicles waiting for trains to clear the at-grade crossing. The reduction in emissions will be most significant during the peak travel periods, although there will be some reduction in emissions throughout the remainder of the day.

1. It is assumed that Average Daily Traffic (ADT) counts be available for the given facility, or facility segments, to be analyzed. This data may be found on highway inventory worksheets or in other data sources if available. Since most vehicle-related air pollution emissions occur during the peak travel periods, the analysis will be conducted for peak and non-peak travel periods. It is assumed that in a typical day there are 4 hours of peak period travel and 20 hours of non-peak travel.

2. If peak hour traffic counts are not available directly, then they may be estimated for both directions (one in the case of single lane roads) using the factors in Table 1 as shown below:

$$\text{PHV} = \left(\begin{matrix}\text{Fraction of traffic} \\ \text{in both directions}\end{matrix}\right) \times \text{ADT}$$

where:

PHV = peak hour volume in both directions

For the four hour peak travel period it is assumed that (4) x (PHV) vehicles pass through the grade crossing. During the remainder of the day, ADT - (4) x (PHV) vehicles pass through the grade crossing.

3. Ideally, it would be desirable to know the average number of trains passing through the grade crossing during the peak and non-peak travel periods. If such data is not available, then an estimate of the total number of trains T passing through the grade crossing is needed. In the absence of any detailed train schedule information, it may be assumed that the number of trains passing through the crossing is uniformly distributed over the day. Therefore,

$$T_p = \frac{T}{6}$$

$$\text{and } T = T_p + T_{np}$$

$$T_{np} = \frac{5}{6} T$$

where:

T = average daily number of trains passing through the crossing

T_p = average daily number of trains passing through the crossing during the peak travel period

T_{np} = average daily number of trains passing through the crossing during the non-peak travel period.

Where T_p and T_{np} may be estimated from actual data, then these values should be used.

4. In order to estimate how long a grade crossing is blocked by a train, it is necessary to know the approximate train length and train speed as it approaches and passes through the crossing. The time interval during which the train blocks the tracks may be estimated as:

$$t_n = \frac{\text{train length}}{\text{train speed}} = \frac{\text{(number of cars)(length per car)}}{\text{(train speed)}}$$

If specific train length and train speed data is not available, the notes on pages V-6 and V-7 can be used to estimate this data.

5. It is assumed that vehicle traffic arrives uniformly at the grade crossing within each travel period (peak and non-peak). Then,

- peak travel period = (4) x (PHV)
- non-peak travel period traffic = ADT - (4) (PHV)
- total train delay during the peak travel period = $T_p \times t_n$
- total train delay during the non-peak travel period = $T_{np} \times t_n$
- total number of vehicles blocked during the peak travel period =

$$\left[\begin{array}{l}\text{fraction of peak travel period}\\ \text{that crossing is blocked by trains}\end{array}\right] \times \left[\text{peak travel period traffic}\right] =$$

$$\left\{\frac{\left(\begin{array}{l}\text{no. of trains in}\\ \text{peak travel period}\end{array}\right) \times \left(\begin{array}{l}\text{time interval}\\ \text{each train}\\ \text{blocks the}\\ \text{crossing}\end{array}\right)}{\text{four hours in peak travel period}}\right\} \times \left[\text{peak travel period traffic}\right] =$$

$$\left(\frac{T_p \times t_n}{4}\right) \times (4\ \text{PHV}) = \boxed{(T_p) \times (t_n) \times (\text{PHV})}$$

- total number of vehicles blocked during the non-peak travel period =

$$\begin{bmatrix}\text{fraction of non-peak travel}\\ \text{period that crossing is}\\ \text{blocked by trains}\end{bmatrix} \times \begin{bmatrix}\text{non-peak travel}\\ \text{period traffic}\end{bmatrix} =$$

$$\left\{\frac{\begin{pmatrix}\text{no. of trains}\\ \text{in peak travel}\\ \text{period}\end{pmatrix} \times \begin{pmatrix}\text{time interval}\\ \text{each train}\\ \text{blocks the}\\ \text{crossing}\end{pmatrix}}{\text{twenty hours in peak travel period}}\right\} \times \begin{bmatrix}\text{non-peak travel}\\ \text{period traffic}\end{bmatrix} =$$

$$\left(\frac{T_{np} \times t_n}{20}\right) \times \left(ADT - 4\ (PHV)\right) = \boxed{\frac{(T_{np}) \times (t_n) \times (ADT - (4)\ (PHV))}{20}}$$

During the time interval t_n, the time that the train blocks the crossing, it is assumed that vehicles arrive uniformly. This means that each vehicle would, on the average, experience a delay of $t_n/2$.

6. By building a grade separated railroad crossing, the net reduction in vehicle idling delay may be estimated by:

$$\begin{matrix}\text{Reduction in peak}\\ \text{travel period ve-}\\ \text{hicle idling de-}\\ \text{lay}\end{matrix} = \begin{pmatrix}\text{total number of}\\ \text{vehicles blocked}\\ \text{during the peak}\\ \text{travel period}\end{pmatrix} \times \begin{pmatrix}\text{average delay}\\ \text{per vehicle}\end{pmatrix}$$

$$= (T_p) \times (t_n) \times (PHV) \times (t_n/2)$$

$$= \frac{T_p \times t_n^2 \times (PHV)}{2}$$

$$\begin{matrix}\text{Reduction in non-}\\ \text{peak travel period}\\ \text{vehicle idling delay}\end{matrix} = \begin{pmatrix}\text{total number of}\\ \text{vehicles blocked}\\ \text{during the non-}\\ \text{peak travel}\\ \text{period}\end{pmatrix} \times \text{(average delay per vehicle)}$$

$$= \frac{(T_{np}) \times (t_n) \times (ADT - (4)\ (PHV))}{20} \times (t_n/2)$$

$$= \frac{T_{np} \times t_n^2 \times (ADT - (4)\ (PHV))}{40}$$

Please note all calculations carried out in hours.

Reduction in total vehicle idling delay

$$= \frac{t_n^2}{2} \left[T_p \times (PHV) + \frac{T_{np} \times (ADT - (4)\ (PHV))}{20} \right]$$

7. The net reduction in vehicle idling emissions during the peak travel period, non-peak travel period and daily are as follows:

Reduction in daily peak travel period emissions related to vehicle idling

$$= \frac{T_p \times t_n^2 \times (PHV)}{2} \times \frac{\text{(automobile emissions)}}{\text{(vehicle-hour of idling)}}$$

Reduction in non-peak travel period emissions related to vehicle idling

$$= \frac{T_{np} \times t_n^2 \times (ADT - (4)\ (PHV))}{2} \times \frac{\text{(automobile emissions)}}{\text{(vehicle-hours of idling)}}$$

Reduction in daily vehicle emissions related to vehicle idling

$$= \frac{t_n^2}{2} \left[T_p \times (PHV) + T_{np} \times (ADT - (4)\ (PHV)) \right] \times \frac{\text{(automobile emissions)}}{\text{(vehicle-hours of idling)}}$$

8. It can be shown that vehicles approaching railroad grade crossings slow down (by 11 MPH on average)* to cross. Using Figures 3, 4 and 5, the additional emissions caused by these speed changes may be estimated. The reduction of these emissions resulting from a railroad grade crossing would be as follows:

*Data from E. St. Louis Railroad-Highway Conflict Demonstration Project Before Study; Hurst-Rosche Engineers, Inc.

$$\begin{array}{l}\text{Reduction in}\\ \text{emissions due}\\ \text{to elimination}\\ \text{of speed changes}\end{array} = \left\{\left[\begin{array}{l}\text{vehicles delayed}\\ \text{at crossing dur-}\\ \text{ing a given travel}\\ \text{period}\end{array}\right] \times \left[\begin{array}{l}\text{emissions per vehicle}\\ \text{from changing speed}\\ e(S'' \rightarrow 0 \rightarrow S'')\end{array}\right]\right\}$$

$$+ \left\{\left[\begin{array}{l}\text{remaining vehicles}\\ \text{not delayed at}\\ \text{crossing during}\\ \text{a given travel}\\ \text{period}\end{array}\right] \times \left[\begin{array}{l}\text{emissions per vehicle}\\ \text{from changing speed from}\\ e(S'' \rightarrow S \rightarrow S'')\end{array}\right]\right\}$$

Definitions of $e(S'' \rightarrow S \rightarrow S'')$ and $e(S'' \rightarrow 0 \rightarrow S'')$ may be found page VI-5.

During the peak travel period the addition of a grade crossing will result in the following decrease in emissions:

$$\begin{array}{l}\text{Decrease in peak}\\ \text{period emis-}\\ \text{sions due to}\\ \text{elimination of}\\ \text{speed changes}\end{array} = \left\{T_p \times t_n \times PHV \times \left[\quad e(S'' \rightarrow 0 \rightarrow S'') \quad\right]\right\} + \left\{(4 \times PHV - T_p \times t_n \times PHV) \times \left[\quad e(S'' \rightarrow S \rightarrow S'') \quad\right]\right\}$$

Similarly, during the non-peak period the following decrease in emissions would occur:

$$\begin{array}{l}\text{Decrease in non-}\\ \text{peak period emis-}\\ \text{sions due to}\\ \text{elimination of}\\ \text{speed changes}\end{array} = \left(\frac{T_{np} \times t_n \times (ADT - (4)\ (PHV))}{20}\right) \times \left[\quad e(S'' \rightarrow 0 \rightarrow S'') \quad\right]$$

$$+ \left((ADT - 4\ \ (PHV))\right)\left(1 - \frac{T_{np} \times t_n}{20}\right) \times \left[\quad e(S'' \rightarrow S \rightarrow S'') \quad\right]$$

Please note that emissions per vehicle per speed change may be determined from Figure 3, 4 and 5 by dividing the numbers of the vertical axis by 1,000.

9. Summary

$$\text{Reduction in total emissions per day} = \underbrace{\left[\frac{t_n{}^2}{2} \times \left[T_p \times (\text{PHV}) + T_{np} \times (\text{ADT} - (4)\ (\text{PHV})\right] \times (\text{automobile emissions per vehicle hour idling})\right]}_{\text{Reduction in idling emissions due to elimination of vehicle delay}}$$

$$+ \underbrace{\left[T_p \times t_n \times \text{PHV} \times e(S'' \rightarrow 0 \rightarrow S'') + (4 \times \text{PHV} - T_p \times t_n \times \text{PHV}) \times \left(e(S'' \rightarrow S \rightarrow S'')\right)\right]}_{\text{Reduction in emissions during peak period from elimination of speed changes}}$$

$$+ \underbrace{\left[\left(\frac{T_{np} \times t_n \times (\text{ADT} - (4)\ (\text{PHV})}{20}\right) \times \left(e(S'' \rightarrow 0 \rightarrow S'')\right) + \left(\text{ADT} - (4)\ (\text{PHV})\right) \times \left(1 - \frac{T_{np} \times t_n}{20}\right) \times \left(e(S'' \rightarrow S \rightarrow S'')\right)\right]}_{\text{Reduction in emissions during non-peak period from elimination of speed changes}}$$

- n - Table 2 and Figure 1 give the distribution of crossing by total number of train movements per day nationwide. The average number of daily train movements ranges between 0 and twenty-five trains per day and then falls off rapidly. It is likely that at locations where a grade separation is being planned that either ADT is very high and/or train movements are frequent.
- Train Speed - Table 3 and Figure 2 show the distribution of maximum train timetable speeds over grade crossings. The most typical speeds range from 6 to 40 mph, however, there are grade crossings which are traversed at much higher speeds. For urban operations, a 20 to 30 mph speed may be assumed if no specific data is available.

- Train Length - According to the 1980 Edition Yearbook of Railroad Facts (American Association of Railroads), the average number of cars per train in the Eastern District (includes St. Louis) was 67.7 in 1979. Assuming that the average car length plus coupler length is about 55 feet, then the average train would be 67.7 cars x 55 feet per car = 3,723.5 feet or 0.71 miles.

  Therefore, if an average length train traveled at a crossing speed of 20 mph it would take 0.0355 hours or 2.13 minutes to clear the crossing.

TABLE 1

PERCENTAGE OF AADT IN PEAK HOUR FOR ONE DIRECTION AND BOTH DIRECTIONS BY PEAK HOUR, 30TH HIGHEST HOUR, AND 200TH HIGHEST HOUR, BY TYPE OF FACILITY

TYPE OF FACILITY	PERCENTAGE OF AADT IN PEAK HOURS					
	ONE DIRECTION			BOTH DIRECTIONS		
	PEAK HOUR	30TH HIGHEST HOUR	200TH HIGHEST HOUR	PEAK HOUR	30TH HIGHEST HOUR	200TH HIGHEST HOUR
Rural:						
Freeway	23.6	15.4	11.4	18.3	13.5	10.9
Expressway	21.5	14.1	10.6	19.2	12.7	9.7
Highway with more than 2 lanes	21.2	13.7	10.3	16.4	12.7	9.9
2-Lane two-way highway	—	—	—	19.7	13.6	11.2
Urban:						
Freeway	15.0	12.7	10.7	13.6	11.0	9.6
Expressway	14.6	11.4	8.9	11.6	9.5	8.3
Street with more than 2 lanes	13.8	11.1	9.6	12.0	10.0	8.7
2-Lane two-way street	—	—	—	13.4	10.6	9.0

Source: 1965 Highway Capacity Manual, Table 3.12, p. 40.

TABLE 2. CROSSINGS BY TOTAL NUMBER OF TRAIN MOVEMENTS PER DAY

NO. TRAINS	NC. XINGS
<1	38202
1-2	66739
3-5	34531
6-10	36678
11-15	12773
16-20	12689
21-25	5536
>25	12014

NO. TRAINS	NO. XINGS	NC. TRAINS	NO. XINGS
01-05	101270	101-105	25
06-10	36678	106-110	60
11-15	12773	111-115	5
16-20	12689	116-120	10
21-25	5536	121-125	13
26-30	4568	126-130	5
31-35	2190	131-135	6
36-40	1698	136-140	12
41-45	540	141-145	8
46-50	1143	146-150	3
51-55	263	151-155	2
56-60	456	156-160	4
61-65	264	161-165	2
66-70	150	166-170	5
71-75	110	171-175	0
76-80	188	176-180	3
81-85	40	181-185	1
86-90	41	186-190	2
91-95	75	191-195	2
96-100	87	196-200	24
<1	38202	>200	9

Source: U.S. Department of Transportation, Federal Railroad Administration, Summary Statistics Of the National Railroad-Highway Crossing Inventory For Public At Grade Crossings, June 1977.

TABLE 3. CROSSINGS BY MAXIMUM TIMETABLE SPEED

SPEED	NO. XINGS.	SPEED	NO. XINGS.	SPEED	NO. XINGS.
01-05	9918	36-40	20348	71-75	872
06-10	31567	41-45	5646	76-80	5289
11-15	17359	46-50	18306	81-85	11
16-20	18019	51-55	2697	86-90	857
21-25	25725	56-60	11814	91-95	0
26-30	28271	61-65	1153	96-100	5
31-35	16178	66-70	4544	>100	5

Source: U.S. Department of Transportation, Federal Railroad Administration, Summary Statistics of the National Railroad-Highway Crossing Inventory For Public At Grade Crossings, June 1977.

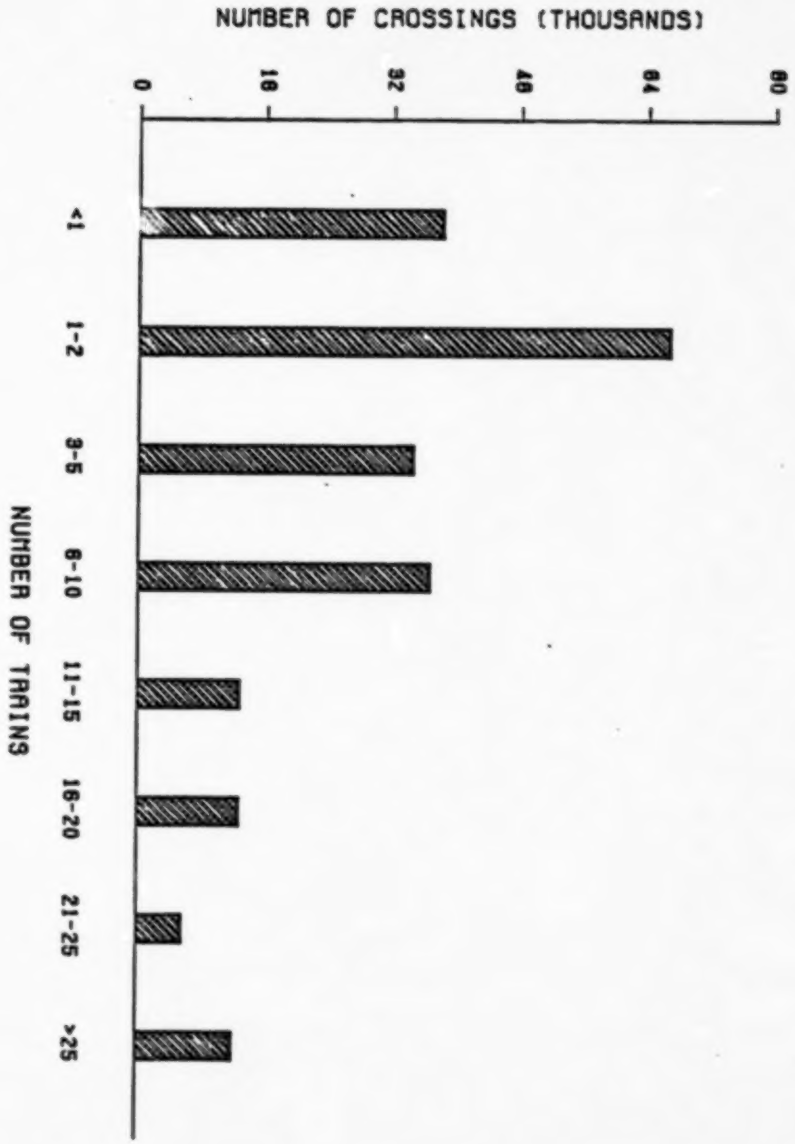


FIGURE 1. CROSSINGS BY TOTAL NUMBER OF TRAIN MOVEMENTS PER DAY

Source: U.S. Department of Transportation, Federal Railroad Administration, Summary Statistics Of the National Railroad-Highway Crossing Inventory For Public At Grade Crossings, June 1977.

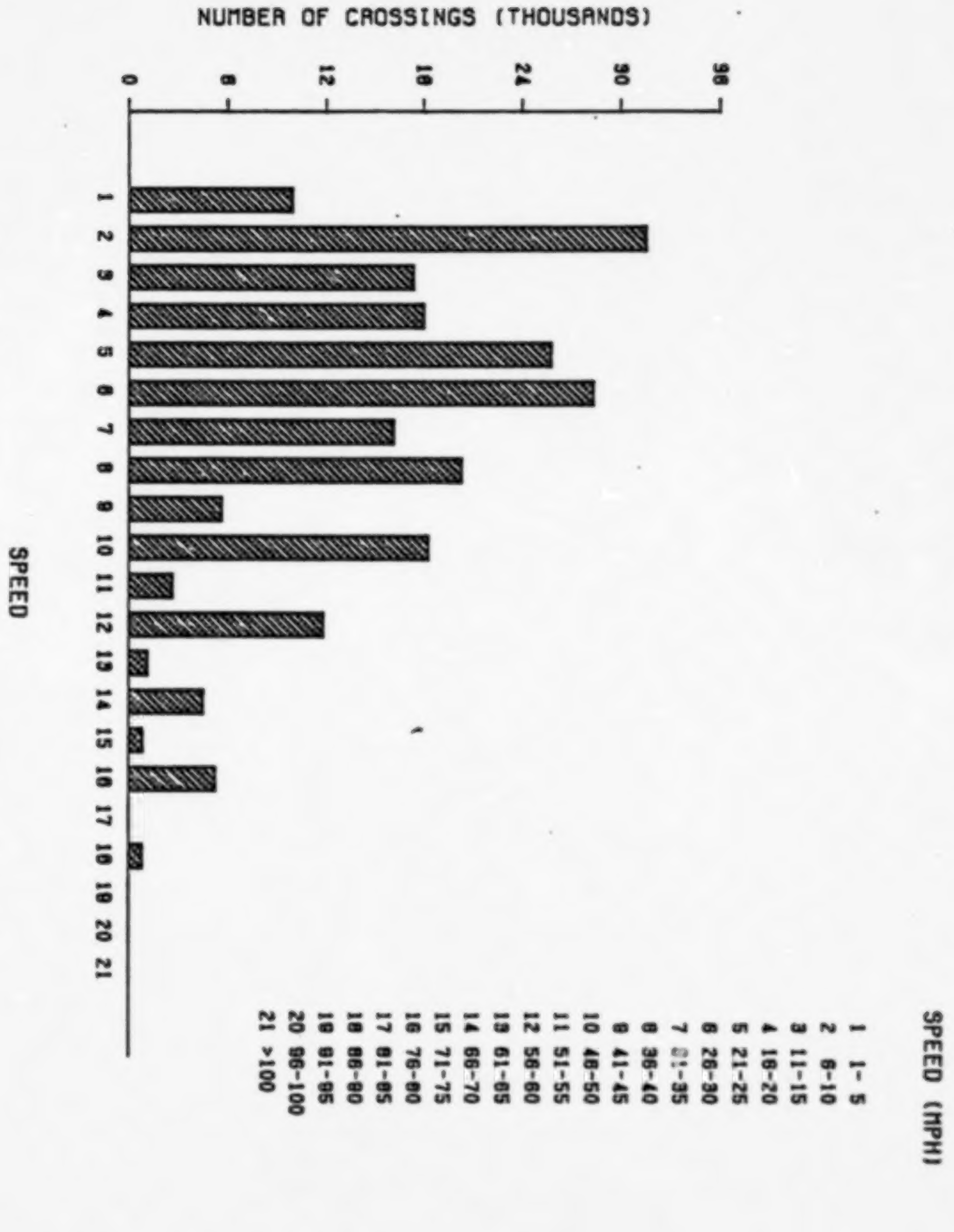
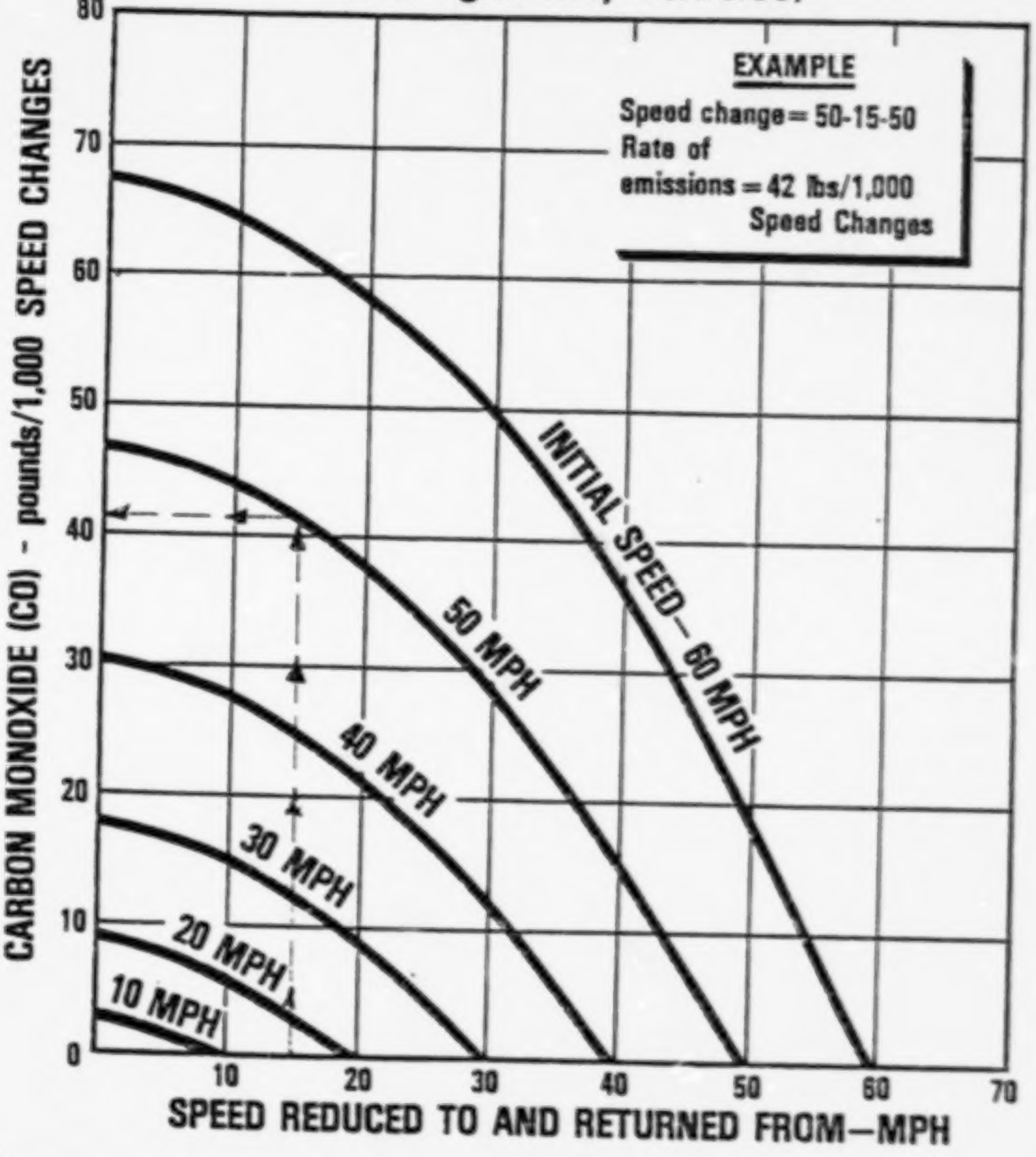


FIGURE 2. CROSSINGS BY MAXIMUM TIMETABLE SPEED

Source: U.S. Department of Transportation, Federal Railroad Administration, Summary Statistics Of the National Railroad-Highway Crossing Inventory For Public At Grade Crossings, June 1977.

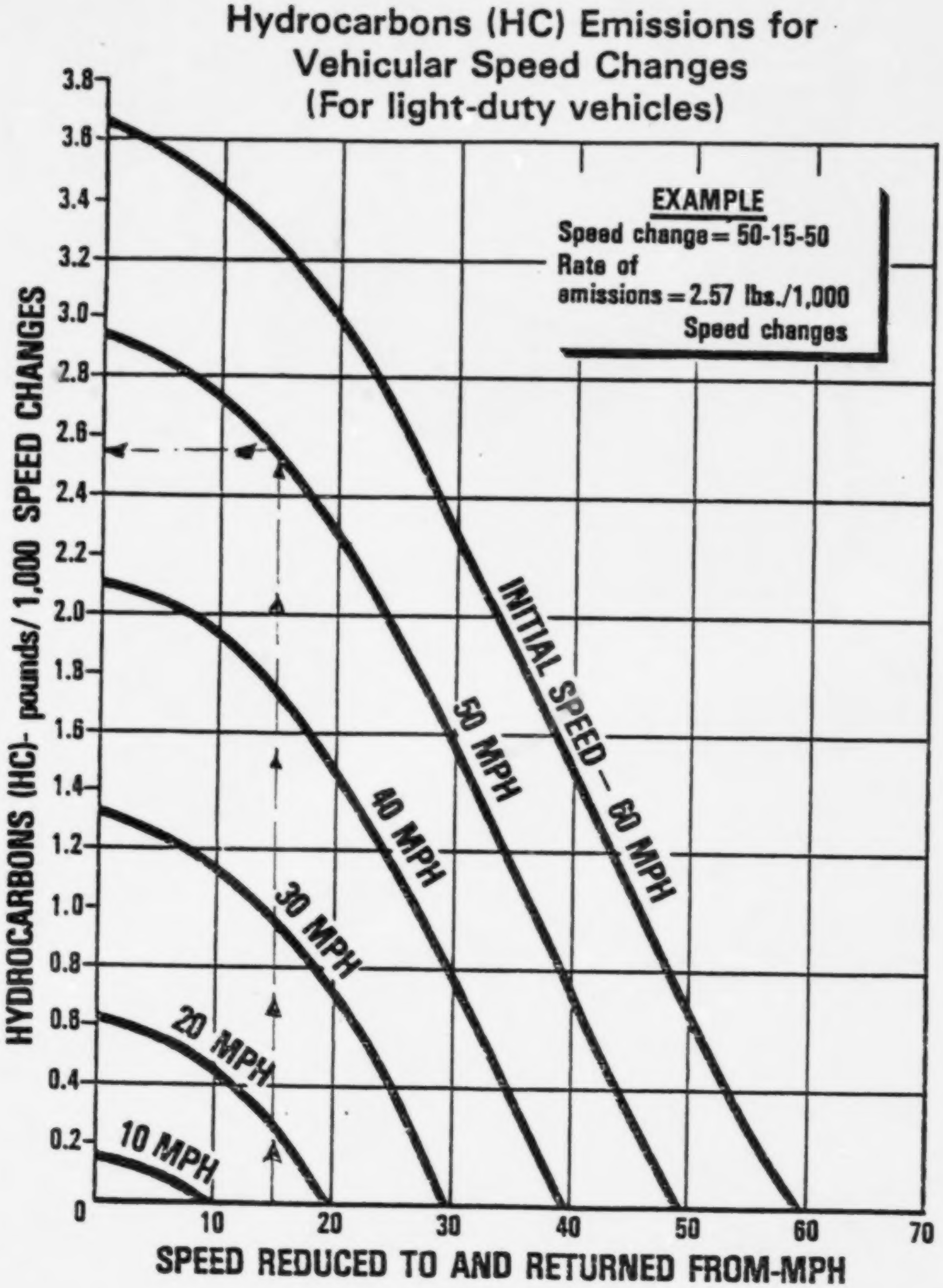
FIGURE 3

Carbon Monoxide (CO) Emissions for Vehicular Speed Changes (For light-duty vehicles)



Source: U.S. Department of Transportation, Federal Highway Administration, Procedure for Estimating Highway User Costs, Fuel Consumption and Air Pollution, March, 1980.

FIGURE 4

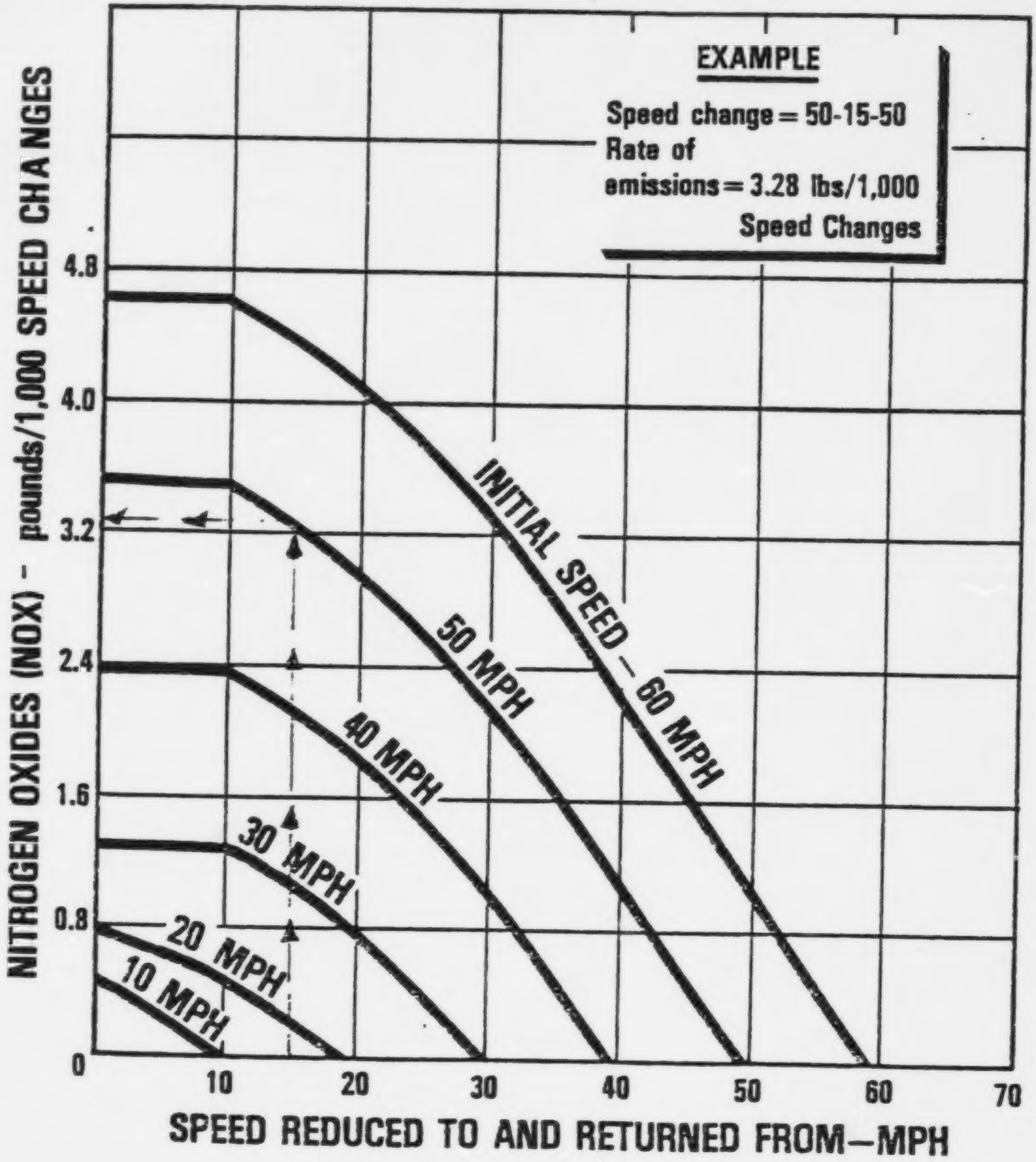


Hydrocarbons (HC) Emissions for Vehicular Speed Changes (For light-duty vehicles)

Source: U.S. Department of Transportation, Federal Highway Administration, Procedure for Estimating Highway User Costs, Fuel Consumption and Air Pollution, March, 1980.

FIGURE 5

Nitrogen Oxides (NOX) Emissions For Vehicular Speed Changes (For light-duty vehicles)



Source: U.S. Department of Transportation, Federal Highway Administration, Procedure for Estimating Highway User Costs, Fuel Consumption and Air Pollution, March, 1980.

VI

PROCEDURES FOR ANALYZING TRANSPORTATION CONTROL MEASURES

WIDEN ROADS/BRIDGES[1]

Lane widths which are narrower than 12 feet have a restrictive effect on traffic. Vehicles are forced to travel closer to each other (laterally) than is normal. Drivers compensate for this by driving more carefully and allowing greater longitudinal spacing between vehicles. Lateral obstructions produce a similar effect.

1. It is assumed that Average Daily Traffic (ADT) counts are available for the given facility, or facility segment, to be analyzed. This data may be found on highway inventory worksheets or other data sources if available. It is also assumed that most congestion, if not all, occurs during the peak travel periods and that during a typical weekday there are 4 peak travel hours and 20 non-peak travel hours. Although the analysis approach presented may be applied during non-peak travel hours, it may be assumed that speed impacts during non-peak travel periods are relatively negligible.

2. In order to perform the analysis, peak hour traffic counts or data on the percent of ADT which occurs during the peak travel hour is required. If such data is not directly available, factors from Table 1 may be used as a proxy and multiplied by ADT to obtain peak hour travel counts. In general, the analysis must be carried out separately for each direction. Therefore, a Directional Factor (DF) which indicates the fraction of traffic in each direction must be applied to the peak hour travel volume. Directional factors may be obtained from highway inventory sheets or, if such information is not available, from "typical" combined directional and peak hour factors as shown in Table 1 for the appropriate type of facility (freeway, expressway, etc.).

$$\begin{aligned} \text{DPHV} &= (\text{ADT}) \times \left(\begin{matrix}\text{fraction of ADT}\\ \text{in peak hour}\end{matrix}\right) \times (\text{DF}) \\ &= (\text{ADT}) \times (\text{CDF}) \end{aligned}$$

where:

DPHV = directional peak hour volume
ADT = average daily traffic
DF = directional factor
CDF = combined directional/peak hour factor

---

[1] Based on methods found in the Highway Capacity Manual (Highway Research Board, Special Report 87, 1965) and Interim Materials on Highway Capacity (Transportation Research Board, Transportation Research Circular No. 212, January 1980).

3. If vehicle count data is available in 15 minute intervals, then the peak hour factor (PHF) may be calculated, for each direction:

$$\text{PHF}_1 = \frac{\text{DPHV}_1}{4 \times (\text{peak rate of flow in 15 minutes})}$$

where: DPHV_1 = directional peak hour volume in 1^{th} direction

PHF_1 = directional peak hour factor in 1^{th} direction

If 15 minute count data is not available for a particular facility or facility segment, PHF factors for a similar type of facility in the same local area may be used. Otherwise a default value between 0.85 and 0.90 may be used.[1]

4. Convert the actual volume (demand) to the SV (service volume) by dividing by the PHF as determined in Step 3.[2]

$$\text{SV}_1 = \frac{(\text{demand volume in } 1^{th} \text{ direction})}{(\text{PHF}_1)} = \frac{\text{DPHV}_1}{\text{PHF}_1} \text{ for the peak travel hour}$$

5. In order to simplify the analysis procedure, it is assumed that buses, large trucks and recreational vehicles comprise a negligible percentage of traffic.[3] In order to calculate the change in average travel speed during the peak travel hour (or any other defined period) the following process should be followed:

A. Look up W, the correction factor for the combined effect of restricted lane width and lateral lane clearance on Table 2 (for expressways and freeways), Table 3 (for multilane rural highways without access control) and Table 4 (two lane highways). The ideal for 12 foot wide highways corresponds to a value of 1.00. Select the table values of W corresponding most closely to the known highway characteristics (lane width, distance from edge of travelled way to obstruction) or interpolate between table values as needed. Determine W for the current lane width prior to widening and W', the correction factor for the new or planned lane width.

---

[1] 1965 Highway Capacity Manual, p. 119.

[2] SV is defined as the service volume under prevailing traffic and roadway conditions for a given Level of Service, in order to apply during non-peak travel hours an analogous method way be applied using a different value for the demand volume and PHF_1.

[3] There are procedures to analyze situations where trucks, buses and recreational vehicles are in the traffic stream outlined in the Highway Capacity Manual 1965 and Interim Materials on Highway Capacity, which involve converting these vehicles to passenger car equivalents (pces). However, this would require more data on terrain and grades and would complicate the analysis. For current level of effort, it is probably not necessary to estimate the impact of trucks, buses and recreational vehicles on traffic flow.

B. Estimate the MSV (maximum service volume under ideal conditions) for both before and after the highway facility is widened:

$$MSV_1 = \frac{SV_1}{W}$$

$$MSV_1' = \frac{SV_1}{W'}$$

where: MSV_1 = maximum service volume before widening in direction 1

MSV_1' = maximum service volume after widening in direction 1

C. Highway speeds may be estimated using capacity and speed parameters for specific facility and area types from the UROAD link speed and capacity tables modified for the St. Louis area. (See Table 5).

Facility Type Codes:

1. freeways
2. expressways
3. arterials (2 way with parking)
4. arterials (1 way)
5. centroid (local and collector streets aggregated)
6. arterial (2 way without parking)

Area Type:

1. Central Business District (CBD)
2. Fringe
3. Residential
4. Outer Central Business District
5. Rural

Table 2 yields the mean free (non-congested) speed and the possible capacity per lane for a specific highway facility and area type. It is assumed that these "possible" capacities are analogous to those developed in the 1965 Highway Capacity Manual. Then

$$S_1 = \frac{So}{1 + 0.15\,(v/c)^4} = \frac{So}{1 + 0.15\,\left(\frac{MSV_1}{C}\right)^4}$$

$$S_1' = \frac{So}{1 + 0.15\,(v/c)^4} = \frac{So}{1 + 0.15\,\left(\frac{MSV_1'}{C}\right)^4}$$

where: S_1 = average speed in direction 1 before road widened

S_1' = average speed in direction 1 after road widened

So = mean free speed for specific facility and area type

C = "practical capacity" = 0.75 x ("possible" capacity per lane from Table 2) x (number of lanes in one direction)

6. Most improvements in air quality due to speed or traffic flow improvements are likely to occur during the peak travel periods. It is assumed that there are 4 hours each day during which peak hour traffic conditions exist. Then the reduction in pollution emissions may be estimated as follows:

Emissions prior to widening $= \sum_1 4 \times DPHV_1 \times e\ (S_1) \times d$

Emissions after widening $= \sum_1 4 \times DPHV_1 \times e(S_1)' \times d$

where: $DPHV_1$ = peak hour traffic volume in direction 1

S_1 = speed in direction 1 prior to widening

S_1' = speed in direction 1 after widening

$e(S_1)'$ = emissions per mile at speed S_1

$e(S_1')$ = emissions per mile at speed S_1'

d = length of road on road segment

$\sum_1$ = sum over each direction 1

If directions are labeled 1 and 2, then:

Emissions prior to widening $= 4 \times d \times \left[DPHV_1 \times e(S_1) + DPHV_2 \times e(S_2)\right]$

Emissions after widening $= 4 \times d \times \left[DPHV_1 \times e(S_1') + DPHV_2 \times e(S_2')\right]$

Net Reduction in emissions $= 4 \times d \times [\ DPHV_1 \times (e(S_1) - e(S_1')) + DPHV_2 \times (e(S_2) - e(S_2'$

7. Upon approaching a bridge (or roadway segment) which is narrower than the connecting road, a vehicle may be forced to slow down because of the bridge. This situation is most likely to occur during the peak period. If the bridge (or roadway segment) is found to have a speed much lower than that of the connecting roadway prior to widening, then the following additional reduction in emissions is likely to occur.

$$\text{Net reduction in emissions due to speed changes} = 4 \times \left[DPHV_1 \times e\,(S_1'' \rightarrow S_1 \rightarrow S_1'') + DPHV_2 \times e\,(S_2'' \rightarrow S_2 \rightarrow S_2'') \right]$$

Where:

S_1'' = speed in direction 1 on roadway approaching the bridge (or roadway segment) prior to widening

S_2'' = speed in direction 2 on roadway approaching the bridge (or roadway segment) prior to widening

S_1 = speed in direction 1 on bridge (or roadway segment) prior to widening

S_2 = speed in direction 2 on bridge (or roadway segment) prior to widening

$e\,(S_1'' \rightarrow S_1 \rightarrow S_1'')$ = notation indicating emissions per vehicle as a result of changing speeds from S_1'' to S_1 and back to S_1''

$e\,(S_2'' \rightarrow S_2 \rightarrow S_2'')$ = notation indicating emissions per vehicle as a result of changing speeds from S_2'' to S_2 and back to S_2''

Figures 1, 2 and 3 may be used to determine the values of $e\,(S_1'' \rightarrow S_1 \rightarrow S_1'')$ and $e\,(S_2'' \rightarrow S_2 \rightarrow S_2'')$. Please note that values on the vertical axis should be divided by 1,000 so as to obtain emissions per single speed change cycle.

TABLE 1

PERCENTAGE OF AADT IN PEAK HOUR FOR ONE DIRECTION AND BOTH DIRECTIONS BY PEAK HOUR, 30TH HIGHEST HOUR, AND 200TH HIGHEST HOUR, BY TYPE OF FACILITY

TYPE OF FACILITY	PERCENTAGE OF AADT IN PEAK HOURS					
	ONE DIRECTION			BOTH DIRECTIONS		
	PEAK HOUR	30TH HIGHEST HOUR	200TH HIGHEST HOUR	PEAK HOUR	30TH HIGHEST HOUR	200TH HIGHEST HOUR
Rural:						
Freeway	23.6	15.4	11.4	18.3	13.5	10.9
Expressway	21.5	14.1	10.6	19.2	12.7	9.7
Highway with more than 2 lanes	21.2	13.7	10.3	16.4	12.7	9.9
2-Lane two-way highway	—	—	—	19.7	13.6	11.2
Urban:						
Freeway	15.0	12.7	10.7	13.6	11.0	9.6
Expressway	14.6	11.4	8.9	11.6	9.5	8.3
Street with more than 2 lanes	13.8	11.1	9.6	12.0	10.0	8.7
2-Lane two-way street	—	—	—	13.4	10.6	9.0

Source: *1965 Highway Capacity Manual*, Table 3.12, p. 40.

TABLE 2

CORRECTION FACTOR FOR THE COMBINED EFFECT OF RESTRICTED LANE WIDTH AND LATERAL CLEARANCE (W)

Distance From Edge of Traveled Way To Obstruction	Adjustment Factor (W)							
	Obstruction on One Side of One-Direction Roadway				Obstruction on Both Sides of One-Direction Roadway			
	Lane Width							
ft. (m.)	12 ft. (3.7m.)	11 ft. (3.4m.)	10 ft. (3.0m.)	9 ft. (2.7m.)	12 ft. (3.7m.)	11 ft. (3.4m.)	10 ft. (3.0m.)	9 ft. (2.7m.)
4-lane divided highway, (2 lanes each direction)								
≥6 (1.8)	1.00	0.97	0.91	0.81	1.00	0.97	0.91	0.81
5 (1.5)	0.99	0.96	0.90	0.80	0.99	0.96	0.90	0.80
4 (1.2)	0.99	0.96	0.90	0.80	0.98	0.95	0.89	0.79
3 (0.9)	0.98	0.95	0.89	0.79	0.96	0.93	0.87	0.77
2 (0.6)	0.97	0.94	0.88	0.79	0.94	0.91	0.86	0.76
1 (0.3)	0.93	0.90	0.85	0.76	0.87	0.85	0.80	0.71
0 (0.0)	0.90	0.87	0.82	0.73	0.81	0.79	0.74	0.66
6- or 8-lane divided highway, (3 or 4 lanes each direction)								
≥6 (1.8)	1.00	0.96	0.89	0.78	1.00	0.96	0.89	0.78
5 (1.5)	0.99	0.95	0.88	0.77	0.99	0.95	0.88	0.77
4 (1.2)	0.99	0.95	0.88	0.77	0.98	0.94	0.87	0.77
3 (0.9)	0.98	0.94	0.87	0.76	0.97	0.93	0.86	0.76
2 (0.6)	0.97	0.93	0.87	0.76	0.96	0.92	0.85	0.75
1 (0.3)	0.95	0.92	0.86	0.75	0.93	0.89	0.83	0.72
0 (0.0)	0.94	0.91	0.85	0.74	0.91	0.87	0.81	0.70

Source: Interim Materials On Highway Capacity, Transportation Research Circular Number 212, January, 1980, Table 2.3, p. 174.

TABLE 3

COMBINED EFFECT OF LANE WIDTH AND RESTRICTED LATERAL CLEARANCE ON CAPACITY AND SERVICE VOLUME OF UNDIVIDED MULTILANE HIGHWAYS WITH UNINTERRUPTED FLOW

DISTANCE FROM TRAFFIC LANE EDGE TO OBSTRUCTION (FT)	ADJUSTMENT FACTOR,[a] *W*, FOR LATERAL CLEARANCE AND LANE WIDTH							
	OBSTRUCTION ON RIGHT SIDE ONLY, OF ONE-DIRECTION TRAVELED WAY (INCLUDES ALLOWANCE FOR OPPOSING TRAFFIC ON LEFT)				OBSTRUCTIONS ON BOTH SIDES OF ONE-DIRECTION TRAVELED WAY[b,c]			
	12-FT LANES	11-FT LANES	10-FT LANES	9-FT LANES	12-FT LANES	11-FT LANES	10-FT LANES	9-FT LANES
(*a*) 4-LANE UNDIVIDED HIGHWAY, ONE DIRECTION OF TRAVEL								
6	1.00	0.95	0.89	0.77	N.A.	N.A.	N.A.	N.A.
4	0.98	0.94	0.88	0.76	N.A.	N.A.	N.A.	N.A.
2	0.95	0.92	0.86	0.75	0.94	0.91	0.86	N.A.
0	0.88	0.85	0.80	0.70	0.81	0.79	0.74	0.66
(*b*) 6-LANE UNDIVIDED HIGHWAY, ONE DIRECTION OF TRAVEL								
6	1.00	0.95	0.89	0.77	N.A.	N.A.	N.A.	N.A.
4	0.99	0.94	0.88	0.76	N.A.	N.A.	N.A.	N.A.
2	0.97	0.93	0.86	0.75	0.96	0.92	0.85	N.A.
0	0.94	0.90	0.83	0.72	0.91	0.87	0.81	0.70

---

[a] Same adjustments for capacity and all levels of service.

[b] Appropriate for use only where normally undivided roadway is temporarily separated into two roadways by obstructions such as centerline barrier, bridge structural elements, piers, and the like, which are closer than would be the opposing traffic.

[c] N.A. = Not applicable; use adjustment for obstruction on right side only. (In these cases, clearance is temporarily greater than the usual separation from opposing traffic, but adjustment for this temporary improvement is not feasible).

Source: 1965 Highway Capacity Manual, Table 10.2, p. 286.

TABLE 4

COMBINED EFFECT OF LANE WIDTH AND RESTRICTED LATERAL CLEARANCE ON CAPACITY AND SERVICE VOLUMES OF TWO-LANE HIGHWAYS WITH UNINTERRUPTED FLOW

DISTANCE FROM TRAFFIC LANE EDGE TO OBSTRUCTION (FT)	ADJUSTMENT FACTORS W, FOR LATERAL CLEARANCE AND LANE WIDTH[a]															
	OBSTRUCTION ON ONE SIDE ONLY[b]								OBSTRUCTIONS ON BOTH SIDES[b]							
	12-FT LANES		11-FT LANES		10-FT LANES		9-FT LANES		12-FT LANES		11-FT LANES		10-FT LANES		9-FT LANES	
		LEVEL E[c]		LEVEL E[c]		LEVEL E[c]		LEVEL E[c]		LEVEL E[c]		LEVEL E[c]		LEVEL E[c]		LEVEL E[c]
6		1.00		0.88		0.81		0.76		1.00		0.88		0.81		0.76
4		0.97		0.85		0.79		0.74		0.94		0.83		0.76		0.71
2		0.93		0.81		0.75		0.70		0.85		0.75		0.69		0.65
0		0.88		0.77		0.71		0.66		0.76		0.67		0.62		0.58

[a] Adjustment W, given for level E, capacity
[b] Includes allowance for opposing traffic.
[c] Capacity.

Source: 1965 Highway Capacity Manual, Table 10.8, p. 303.

TABLE 5

FLORISSANT 5-ITERATION ASSIGNMENT USING BASENET.CCI8

80CT80   21.40.21   URCAD   REPORT 10

LINK SPEED AND CAPACITY TABLES

AREA TYPE 1

LANES		FACILITY TYPE 1	2	3	4	5	6
1	CAPACITY	1750.	800.	600.	700.	10000.	600
	SPEED	45.0	35.0	20.0	22.0	10.0	22.0
2	CAPACITY	1750.	800.	600.	700.	10000.	600.
	SPEED	45.0	35.0	20.0	22.0	10.0	22.0
3	CAPACITY	1750.	800.	600.	700.	10000.	600.
	SPEED	45.0	35.0	20.0	22.0	10.0	22.0
4	CAPACITY	1750.	800.	600.	700.	10000.	600.
	SPEED	45.0	35.0	20.0	22.0	10.0	22.0
5	CAPACITY	1750.	800.	600.	700.	10000.	600.
	SPEED	45.0	35.0	20.0	22.0	10.0	22.0
6	CAPACITY	1750.	800.	600.	700.	10000.	600.
	SPEED	45.0	35.0	20.0	22.0	10.0	22.0
7	CAPACITY	1750.	800.	600.	700.	10000.	600.
	SPEED	45.0	35.0	20.0	22.0	10.0	22.0
8	CAPACITY	1750.	800.	600.	700.	10000.	600.
	SPEED	45.0	35.0	20.0	22.0	10.0	22.0
9	CAPACITY	1750.	800.	600.	700.	10000.	600.
	SPEED	45.0	35.0	20.0	22.0	10.0	22.0

AREA TYPE 2

LANES		FACILITY TYPE 1	2	3	4	5	6
1	CAPACITY	1750.	1000.	550.	550.	10000.	800.
	SPEED	45.0	40.0	25.0	29.0	15.0	25.0
2	CAPACITY	1750.	1000.	550.	550.	10000.	800.
	SPEED	45.0	40.0	25.0	29.0	15.0	25.0
3	CAPACITY	1750.	1000.	550.	550.	10000.	800.
	SPEED	45.0	40.0	25.0	29.0	15.0	25.0
4	CAPACITY	1750.	1000.	550.	550.	10000.	800.
	SPEED	45.0	40.0	25.0	29.0	15.0	25.0
5	CAPACITY	1750.	1000.	550.	550.	10000.	800.
	SPEED	45.0	40.0	25.0	29.0	15.0	25.0
6	CAPACITY	1750.	1000.	550.	550.	10000.	800.
	SPEED	45.0	40.0	25.0	29.0	15.0	25.0
7	CAPACITY	1750.	1000.	550.	550.	10000.	800.
	SPEED	45.0	40.0	25.0	29.0	15.0	25.0
8	CAPACITY	1750.	1000.	550.	550.	10000.	800.
	SPEED	45.0	40.0	25.0	29.0	15.0	25.0
9	CAPACITY	1750.	1000.	550.	550.	10000.	800.
	SPEED	45.0	40.0	25.0	29.0	15.0	25.0

UNITSO CAPACITY IN %VEH/LANE /HR
SPEED IN MI/HR

TABLE 5
(continued)

AREA TYPE 3

		FACILITY TYPE					
LANES		1	2	3	4	5	6
1	CAPACITY	1750.	1100.	550.	900.	10000.	800.
	SPEED	50.0	40.0	30.0	32.0	15.0	28.0
2	CAPACITY	1750.	1100.	550.	900.	10000.	800.
	SPEED	50.0	40.0	30.0	32.0	15.0	28.0
3	CAPACITY	1750.	1100.	550.	900.	10000.	800.
	SPEED	50.0	40.0	30.0	32.0	15.0	28.0
4	CAPACITY	1750.	1100.	550.	900.	10000.	800.
	SPEED	50.0	40.0	30.0	32.0	15.0	28.0
5	CAPACITY	1750.	1100.	550.	900.	10000.	800.
	SPEED	50.0	40.0	30.0	32.0	15.0	28.0
6	CAPACITY	1750.	1100.	550.	900.	10000.	800.
	SPEED	50.0	40.0	30.0	32.0	15.0	28.0
7	CAPACITY	1750.	1100.	550.	900.	10000.	800.
	SPEED	50.0	40.0	30.0	32.0	15.0	28.0
8	CAPACITY	1750.	1100.	550.	900.	10000.	800.
	SPEED	50.0	40.0	30.0	32.0	15.0	28.0
9	CAPACITY	1750.	1100.	550.	900.	10000.	800.
	SPEED	50.0	40.0	30.0	32.0	15.0	28.0

AREA TYPE 4

		FACILITY TYPE					
LANES		1	2	3	4	5	6
1	CAPACITY	1750.	1000.	550.	650.	10000.	800.
	SPEED	45.0	35.0	20.0	24.0	15.0	22.0
2	CAPACITY	1750.	1000.	550.	650.	10000.	800.
	SPEED	45.0	35.0	20.0	24.0	15.0	22.0
3	CAPACITY	1750.	1000.	550.	650.	10000.	800.
	SPEED	45.0	35.0	20.0	24.0	15.0	22.0
4	CAPACITY	1750.	1000.	550.	650.	10000.	800.
	SPEED	45.0	35.0	20.0	24.0	15.0	22.0
5	CAPACITY	1750.	1000.	550.	650.	10000.	800.
	SPEED	45.0	35.0	20.0	24.0	15.0	22.0
6	CAPACITY	1750.	1000.	550.	650.	10000.	800.
	SPEED	45.0	35.0	20.0	24.0	15.0	22.0
7	CAPACITY	1750.	1000.	550.	650.	10000.	800.
	SPEED	45.0	35.0	20.0	24.0	15.0	22.0
8	CAPACITY	1750.	1000.	550.	650.	10000.	800.
	SPEED	45.0	35.0	20.0	24.0	15.0	22.0
9	CAPACITY	1750.	1000.	550.	650.	10000.	800.
	SPEED	45.0	35.0	20.0	24.0	15.0	22.0

UNITS: CAPACITY IN VEH/LANE /HR
SPEED IN MI/HR

TABLE 5
(continued)

AREA TYPE 5

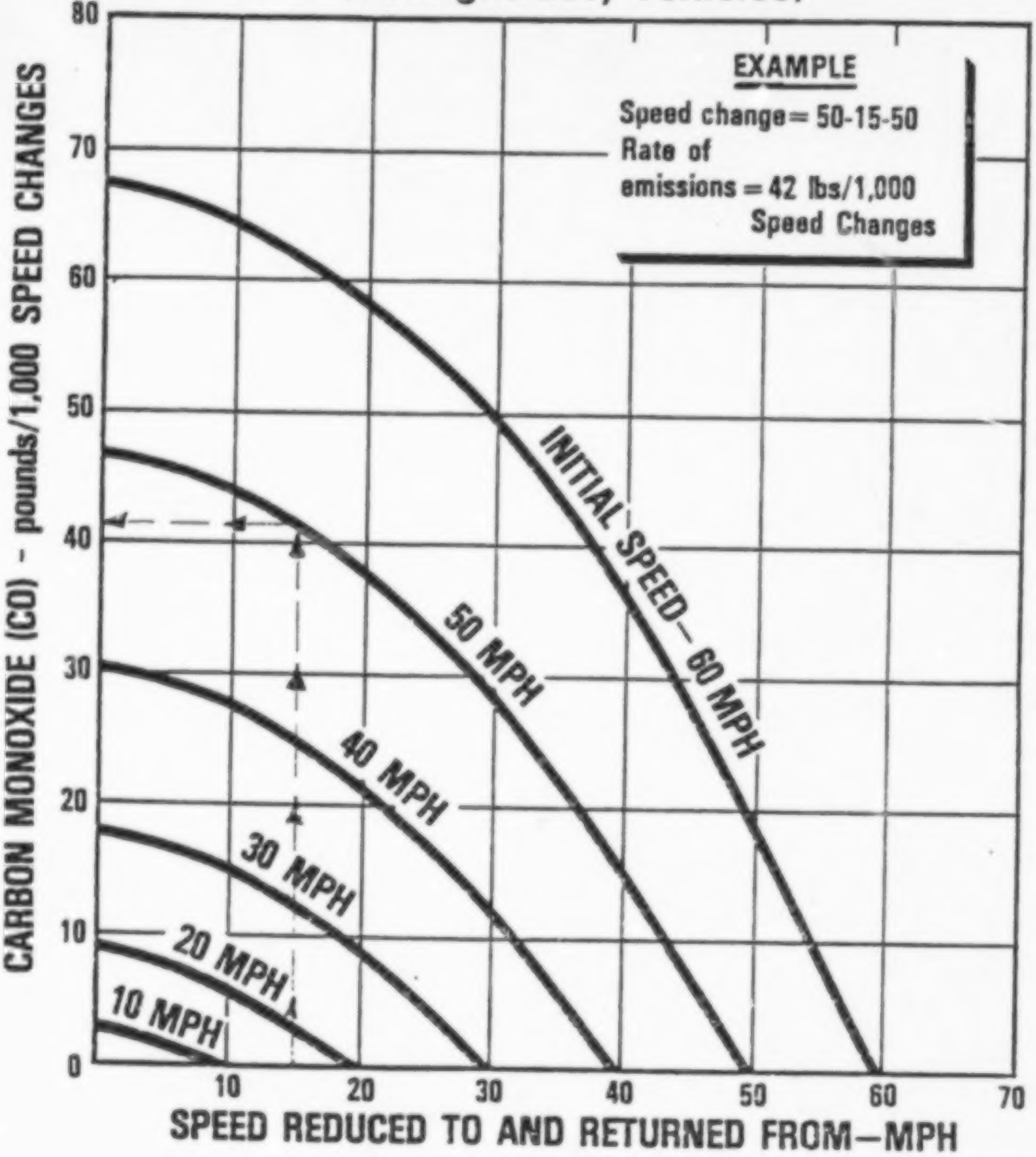
		FACILITY TYPE					
LANES		1	2	3	4	5	6
1	CAPACITY	1750.	1100.	550.	900.	10000.	800.
	SPEED	55.0	45.0	30.0	32.0	15.0	28.0
2	CAPACITY	1750.	1100.	550.	900.	10000.	800.
	SPEED	55.0	45.0	30.0	32.0	15.0	28.0
3	CAPACITY	1750.	1100.	550.	900.	10000.	800.
	SPEED	55.0	45.0	30.0	32.0	15.0	28.0
4	CAPACITY	1750.	1100.	550.	900.	10000.	800.
	SPEED	55.0	45.0	30.0	32.0	15.0	28.0
5	CAPACITY	1750.	1100.	550.	900.	10000.	800.
	SPEED	55.0	45.0	30.0	32.0	15.0	28.0
6	CAPACITY	1750.	1100.	550.	900.	10000.	800.
	SPEED	55.0	45.0	30.0	32.0	15.0	28.0
7	CAPACITY	1750.	1100.	550.	900.	10000.	800.
	SPEED	55.0	45.0	30.0	32.0	15.0	28.0
8	CAPACITY	1750.	1100.	550.	900.	10000.	800.
	SPEED	55.0	45.0	30.0	32.0	15.0	28.0
9	CAPACITY	1750.	1100.	550.	900.	10000.	800.
	SPEED	55.0	45.0	30.0	32.0	15.0	28.0

```
UNITSO CAPACITY IN %VEH/LANE /HR
       SPEED IN MI/HR

UROAD LLL) % INFORMATION 0 188K BYTES OF CORE NEEDED FOR NETWORK INPUT

ROA2 2600 % INFORMATION 0  OF ON OLDHR AT NODE 4040  858 LINKS READ
```

FIGURE 1

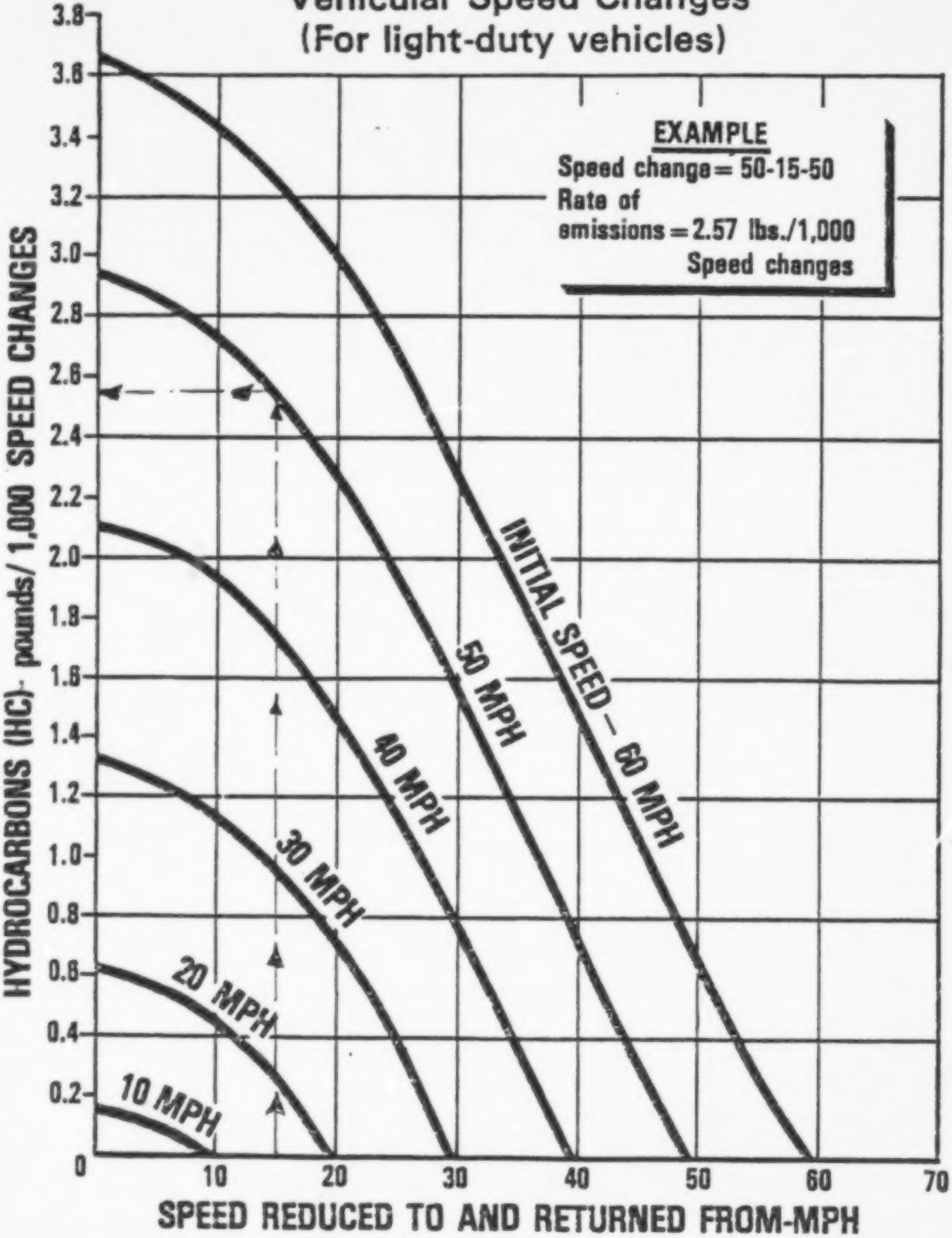


Carbon Monoxide (CO) Emissions
for Vehicular Speed Changes
(For light-duty vehicles)

Source: U.S. Department of Transportation, Federal Highway Administration, Procedure for Estimating Highway User Costs, Fuel Consumption and Air Pollution, March, 1980.

FIGURE 2

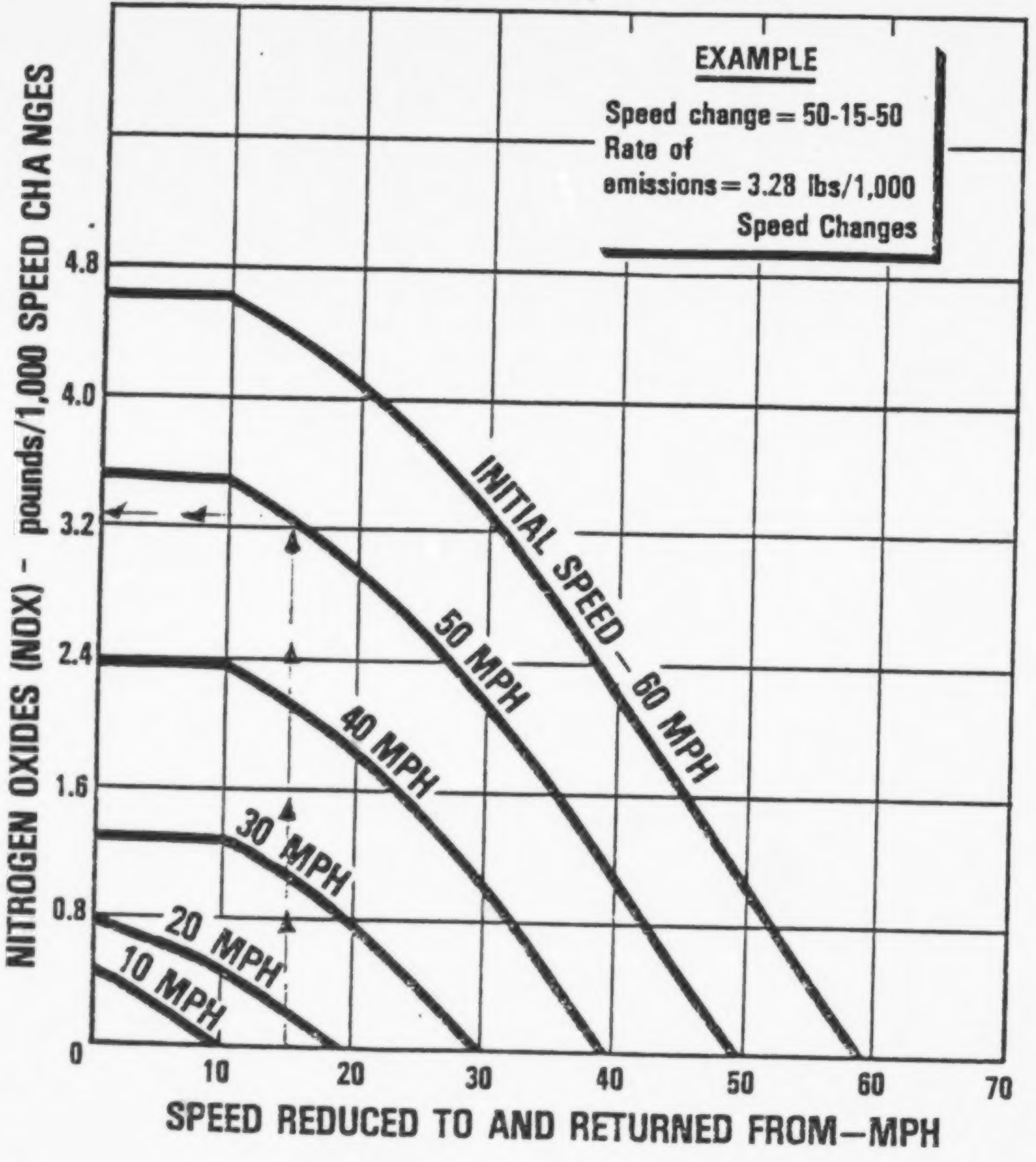
Hydrocarbons (HC) Emissions for Vehicular Speed Changes (For light-duty vehicles)



Source: U.S. Department of Transportation, Federal Highway Administration, Procedure for Estimating Highway User Costs, Fuel Consumption and Air Pollution, March, 1980.

FIGURE 3

Nitrogen Oxides (NOX) Emissions For Vehicular Speed Changes (For light-duty vehicles)



Source: U.S. Department of Transportation, Federal Highway Administration, Procedure for Estimating Highway User Costs, Fuel Consumption and Air Pollution, March, 1980.

VII.

PROCEDURES FOR ANALYZING TRANSPORTATION CONTROL MEASURES

ADD LANES

Adding lanes will generally result in a large increase in capacity and a subsequent increase in average travel speed.

1. It is assumed that Average Daily Traffic (ADT) counts are available for the given facility, or facility segment, to be analyzed. This data may be found on highway inventory worksheets or other data sources if available. It is also assumed that most congestion, if not all, occurs during the peak travel periods and that during a typical weekday there are 4 peak travel hours and 20 non-peak travel hours. Although the analysis approach presented may be applied during non-peak travel hours, it may be assumed that speed impacts during non-peak travel periods are relatively negligible.

2. In order to perform the analysis, peak hour traffic counts or data on the percent of ADT which occurs during the peak travel hour is required. If such data is not directly available, factors from Table 1 may be used as a proxy and multiplied by ADT to obtain peak hour travel counts. In general the analysis must be carried out separately for each direction. Therefore, a Directional Factor (DF) which indicates the fraction of traffic in each direction must be applied to the peak hour travel volume. Directional factors may be obtained from highway inventory sheets or, if such information is not available, from "typical" combined directional and peak hour factors as shown in Table 1 for the appropriate type of facility (freeway, expressway, etc.).

$$\begin{aligned} \text{DPHV} &= (\text{ADT}) \times \left(\begin{array}{l}\text{fraction of ADT}\\ \text{in peak hour}\end{array}\right) \times (\text{DF}) \\ &= (\text{ADT}) \times (\text{CDF}) \end{aligned}$$

where:

DPHV = directional peak hour volume
ADT = average daily traffic
DF = directional factor
CDF = combined directional/peak hour factor

3. If vehicle count data is available in 15 minute intervals, then the peak hour factor (PHF) may be calculated, for each direction:

$$\text{PHF}_1 = \frac{\text{DPHV}_1}{4 \times (\text{peak rate of flow in 15 minutes})}$$

where:

DPHV_1 = directional peak hour volume in 1^{th} direction

PHF_1 = directional peak hour factor in 1^{th} direction

If 15 minute count data is not available for a particular facility or facility segment, PHF factors for a similar type of facility in the same local area may be used. Otherwise a default value between 0.85 and 0.90 may be used.[1]

4. Convert the actual volume (demand) to the SV (service volume) by dividing by the PHF as determined in Step 3.[2]

$$SV_1 = \frac{(\text{demand volume in } 1^{th} \text{direction})}{(PHF_1)} = \frac{DPHV_1}{PHF_1} \text{ for the peak travel hour}$$

5. In order to simplify the analysis, it is assumed that buses, trucks and recreational vehicles comprise a negligible percentage of the total traffic flow. It is also assumed that when lanes are added, they are added in pairs, that is one or more lanes in each direction (i.e., no three lane highways.[3]

A. Highway speeds may be estimated using capacity and speed parameters for specific facility and area types from the UROAD link speed and capacity tables modified for the St. Louis area. (See Table 2).

Facility Type Codes:

1. freeways
2. expressways
3. arterials (2 way with parking)
4. arterials (1 way)
5. centroid (local and collector streets aggregated)
6. arterial (2 way without parking)

Area Type:

1. Central Business District
2. Fringe
3. Residential
4. Outer Central Business District
5. Rural

Table 2 yields the mean free (non-congested) speed and the "possible" capacity per lane for a specific highway facility and area type.

1/ 1965 Highway Capacity Manual, p. 119.

2/ SV is defined as the service volume under prevailing traffic and roadway conditions for a given Level of Service. In order to apply during non-peak travel hours an analogous method way be applied using a different value for the demand volume and PHF_1.

3/ 1965 Highway Capacity Manual, p. 317. Three lane highways are seldom, if ever, designed and built in the U.S. and they are not a logical stage construction step in building a four lane highway.

B. Speeds before and after the new lanes are added may be estimated as follows:

$$S_1 = \frac{So}{1 + 0.15\ (^{v}/_{c})^4} = \frac{So}{1 + 0.15\ (^{SV_1}/_{C})^4}$$

$$S_1' = \frac{So'}{1 + 0.15\ (^{v}/_{c})^4} = \frac{So'}{1 + 0.15\ (^{SV_1}/_{C'})^4}$$

where: S_1 = average speed in direction 1 before new lane added

S_1' = average speed in direction 1 after new lane added

So = mean free speed on facility before new lane added

So' = mean free speed on facility after new lane added

SV_1 = source volume under prevailing traffic and roadway conditions for a given level of service

C = "practical capacity" = 0.75 x ("possible" capacity per lane) x (number of lanes in each direction prior to adding lanes)

C' = "practical capacity" = 0.75 x ("possible" capacity per lane) x (number of lanes in each direction after adding lanes).

6. Most improvements in air quality due to speed on traffic flow improvements are likely to occur during the peak travel periods. It is assumed that these are 4 hours each day where peak hour traffic conditions exist. Then the reduction in air pollution emissions may be estimated as follows:

$$\text{Emissions prior to adding lanes} = \sum_1 4 \times d \times DPHV_1 \times e(S_1)$$

where:

$DPHV_1$ = peak hour traffic volume in direction 1

S_1 = speed in direction 1 prior to adding lanes

$e(S_1)$ = emissions per mile at speed S_1

d = length of road or road segment

$\sum_1$ = sum over each direction 1

If directions are labeled 1 and 2, then:

Emissions prior to adding lanes $= 4 \times d \times \left[DPHV_1 \times e(S_1) + DPHV_2 \times e(S_2) \right]$

Emissions after adding lanes $= 4 \times d \times \left[DPHV_1 \times e(S_1') + DPHV_2 \times e(S_2') \right]$

Net reduction in emissions after adding lanes $= 4 \times d \times \left[DPHV_1 \, (e(S_1) - e(S_1')) + DPHV_2 \, (e(S_2) - e(S_2')) \right]$

Caveats

- This analysis assumes that no new traffic is generated or added to the highway when an extra lane is added.

TABLE 1

PERCENTAGE OF AADT IN PEAK HOUR FOR ONE DIRECTION AND BOTH DIRECTIONS BY PEAK HOUR, 30TH HIGHEST HOUR, AND 200TH HIGHEST HOUR, BY TYPE OF FACILITY

TYPE OF FACILITY	PERCENTAGE OF AADT IN PEAK HOURS					
	ONE DIRECTION			BOTH DIRECTIONS		
	PEAK HOUR	30TH HIGHEST HOUR	200TH HIGHEST HOUR	PEAK HOUR	30TH HIGHEST HOUR	200TH HIGHEST HOUR
Rural:						
Freeway	23.6	15.4	11.4	18.3	13.5	10.9
Expressway	21.5	14.1	10.6	19.2	12.7	9.7
Highway with more than 2 lanes	21.2	13.7	10.3	16.4	12.7	9.9
2-Lane two-way highway	—	—	—	19.7	13.6	11.2
Urban:						
Freeway	15.0	12.7	10.7	13.6	11.0	9.6
Expressway	14.6	11.4	8.9	11.6	9.5	8.3
Street with more than 2 lanes	13.8	11.1	9.6	12.0	10.0	8.7
2-Lane two-way street	—	—	—	13.4	10.6	9.0

Source: 1965 Highway Capacity Manual, Table 3.12, p. 40.

TABLE 2

FLORISSANT 5-ITERATION ASSIGNMENT USING BASENET.CCI8

80CT80 21.40.21 URCAD REPCRT 10

LINK SPEED AND CAPACITY TABLES

AREA TYPE 1

		FACILITY TYPE					
LANES		1	2	3	4	5	6
1	CAPACITY	1750.	800.	600.	700.	10000.	600
	SPEED	45.0	35.0	20.0	22.0	10.0	22.0
2	CAPACITY	1750.	800.	600.	700.	10000.	600.
	SPEED	45.0	35.0	20.0	22.0	10.0	22.0
3	CAPACITY	1750.	800.	600.	700.	10000.	600.
	SPEED	45.0	35.0	20.0	22.0	10.0	22.0
4	CAPACITY	1750.	800.	600.	700.	10000.	600.
	SPEED	45.0	35.0	20.0	22.0	10.0	22.0
5	CAPACITY	1750.	800.	600.	700.	10000.	600.
	SPEED	45.0	35.0	20.0	22.0	10.0	22.0
6	CAPACITY	1750.	800.	600.	700.	10000.	600.
	SPEED	45.0	35.0	20.0	22.0	10.0	22.0
7	CAPACITY	1750.	800.	600.	700.	10000.	600.
	SPEED	45.0	35.0	20.0	22.0	10.0	22.0
8	CAPACITY	1750.	800.	600.	700.	10000.	600.
	SPEED	45.0	35.0	20.0	22.0	10.0	22.0
9	CAPACITY	1750.	800.	600.	700.	10000.	600.
	SPEED	45.0	35.0	20.0	22.0	10.0	22.0

AREA TYPE 2

		FACILITY TYPE					
LANES		1	2	3	4	5	6
1	CAPACITY	1750.	1000.	550.	550.	10000.	800.
	SPEED	45.0	40.0	25.0	29.0	15.0	25.0
2	CAPACITY	1750.	1000.	550.	550.	10000.	800.
	SPEED	45.0	40.0	25.0	29.0	15.0	25.0
3	CAPACITY	1750.	1000.	550.	550.	10000.	800.
	SPEED	45.0	40.0	25.0	29.0	15.0	25.0
4	CAPACITY	1750.	1000.	550.	550.	10000.	800.
	SPEED	45.0	40.0	25.0	29.0	15.0	25.0
5	CAPACITY	1750.	1000.	550.	550.	10000.	800.
	SPEED	45.0	40.0	25.0	29.0	15.0	25.0
6	CAPACITY	1750.	1000.	550.	550.	10000.	800.
	SPEED	45.0	40.0	25.0	29.0	15.0	25.0
7	CAPACITY	1750.	1000.	550.	550.	10000.	800.
	SPEED	45.0	40.0	25.0	29.0	15.0	25.0
8	CAPACITY	1750.	1000.	550.	550.	10000.	800.
	SPEED	45.0	40.0	25.0	29.0	15.0	25.0
9	CAPACITY	1750.	1000.	550.	550.	10000.	800.
	SPEED	45.0	40.0	25.0	29.0	15.0	25.0

UNITSO CAPACITY IN %VEH/LANE /HR
SPEED IN MI/HR

TABLE 2
(continued)

AREA TYPE 3

LANES		FACILITY TYPE					
		1	2	3	4	5	6
1	CAPACITY	1750.	1100.	550.	900.	10000.	800.
	SPEED	50.0	40.0	30.0	32.0	15.0	28.0
2	CAPACITY	1750.	1100.	550.	900.	10000.	800.
	SPEED	50.0	40.0	30.0	32.0	15.0	28.0
3	CAPACITY	1750.	1100.	550.	900.	10000.	800.
	SPEED	50.0	40.0	30.0	32.0	15.0	28.0
4	CAPACITY	1750.	1100.	550.	900.	10000.	800.
	SPEED	50.0	40.0	30.0	32.0	15.0	28.0
5	CAPACITY	1750.	1100.	550.	900.	10000.	800.
	SPEED	50.0	40.0	30.0	32.0	15.0	28.0
6	CAPACITY	1750.	1100.	550.	900.	10000.	800.
	SPEED	50.0	40.0	30.0	32.0	15.0	28.0
7	CAPACITY	1750.	1100.	550.	900.	10000.	800.
	SPEED	50.0	40.0	30.0	32.0	15.0	28.0
8	CAPACITY	1750.	1100.	550.	900.	10000.	800.
	SPEED	50.0	40.0	30.0	32.0	15.0	28.0
9	CAPACITY	1750.	1100.	550.	900.	10000.	800.
	SPEED	50.0	40.0	30.0	32.0	15.0	28.0

AREA TYPE 4

LANES		FACILITY TYPE					
		1	2	3	4	5	6
1	CAPACITY	1750.	1000.	550.	650.	10000.	800.
	SPEED	45.0	35.0	20.0	24.0	15.0	22.0
2	CAPACITY	1750.	1000.	550.	650.	10000.	800.
	SPEED	45.0	35.0	20.0	24.0	15.0	22.0
3	CAPACITY	1750.	1000.	550.	650.	10000.	800.
	SPEED	45.0	35.0	20.0	24.0	15.0	22.0
4	CAPACITY	1750.	1000.	550.	650.	10000.	800.
	SPEED	45.0	35.0	20.0	24.0	15.0	22.0
5	CAPACITY	1750.	1000.	550.	650.	10000.	800.
	SPEED	45.0	35.0	20.0	24.0	15.0	22.0
6	CAPACITY	1750.	1000.	550.	650.	10000.	800.
	SPEED	45.0	35.0	20.0	24.0	15.0	22.0
7	CAPACITY	1750.	1000.	550.	650.	10000.	800.
	SPEED	45.0	35.0	20.0	24.0	15.0	22.0
8	CAPACITY	1750.	1000.	550.	650.	10000.	800.
	SPEED	45.0	35.0	20.0	24.0	15.0	22.0
9	CAPACITY	1750.	1000.	550.	650.	10000.	800.
	SPEED	45.0	35.0	20.0	24.0	15.0	22.0

UNITSO CAPACITY IN %VEH/LANE /HR
SPEED IN MI/HR

TABLE 2
(continued)

AREA TYPE 5

LANES		FACILITY TYPE 1	2	3	4	5	6
1	CAPACITY	1750.	1100.	550.	900.	10000.	800.
	SPEED	55.0	45.0	30.0	32.0	15.0	28.0
2	CAPACITY	1750.	1100.	550.	900.	10000.	800.
	SPEED	55.0	45.0	30.0	32.0	15.0	28.0
3	CAPACITY	1750.	1100.	550.	900.	10000.	800.
	SPEED	55.0	45.0	30.0	32.0	15.0	28.0
4	CAPACITY	1750.	1100.	550.	900.	10000.	800.
	SPEED	55.0	45.0	30.0	32.0	15.0	28.0
5	CAPACITY	1750.	1100.	550.	900.	10000.	800.
	SPEED	55.0	45.0	30.0	32.0	15.0	28.0
6	CAPACITY	1750.	1100.	550.	900.	10000.	800.
	SPEED	55.0	45.0	30.0	32.0	15.0	28.0
7	CAPACITY	1750.	1100.	550.	900.	10000.	800.
	SPEED	55.0	45.0	30.0	32.0	15.0	28.0
8	CAPACITY	1750.	1100.	550.	900.	10000.	800.
	SPEED	55.0	45.0	30.0	32.0	15.0	28.0
9	CAPACITY	1750.	1100.	550.	900.	10000.	800.
	SPEED	55.0	45.0	30.0	32.0	15.0	28.0

```
UNITSO CAPACITY IN %VEH/LANE /HR
       SPEED IN MI/HR

UROAD LLL) % INFORMATION 0 188K BYTES OF CORE NEEDED FOR NETWORK INPUT

ROA2 2600 % INFORMATION 0  OF ON OLDHR AT NODE 4040  858 LINKS READ
```

VIII.

PROCEDURES FOR ANALYZING TRANSPORTATION CONTROL MEASURES
MAJOR NEW FACILITIES

Key highway facilities being planned or in various stages of construction include I-270 and the inner belt (725). Currently traffic travels on parallel routes Illinois 157/159 and the Lindburgh route, respectively.

1. Assuming no change in travel demand, extra capacity provided by a new parallel route will result in lower volume to capacity ratios and increased speeds. The reduction in emissions brought about by more efficient travel speeds will largely occur during the peak travel period, assumed to be four hours on a typical day.

2. The transportation planning group at the East-West Gateway Coordinating Council (EWGCC) will run its transportation models on a 1995 network with major new facilities coded into the network. Model output will include average travel speed and travel demand (and therefore vehicle-miles of-travel (VMT)) on each link along the new highway facilities and existing parallel routes. In order to factor down 1995 model results to 1987 (SIP evaluation year) a travel demand growth factor is needed. If no such factor is available, then it will be assumed that 1995 travel closely approximates that which is likely to exist in 1987.

3. Ideally it would be desirable if EWCGG would re-run the highway network model without the new facilities. Then the difference in peak period travel characteristics (average speed and vehicle-miles of-travel) could be used to estimate the net reduction in vehicle emissions. However, if EWGCC doesn't re-run the highway network model without the new facilities, then the following approach can be used to estimate the reduction in daily peak period emissions:

   A. Air pollution emissions with new highway facilities in place may be calculated as:

$$Ep' = 4\sum_{l}\left[\sum_{k} d_k x (DPHV'_{kl}) \text{ x } e(S'_{kl}) + \sum_{ko} d_{ko} \text{ x } (DPHV'_{kol}) \text{ x } e(S'_{kol})\right]$$

where:

Ep' = daily peak period emissions after new highway facility built

l = direction indicator for each highway facility

k = link number on old facility

k_o = link number on new facility

d = length of link

DPHV = directional peak hour traffic volume

S = average travel speed

e(S) = emissions per vahicle-mile—of-travel at speed S

' = indicates parameters for after new facilities in place

B. Average Daily Traffic (ADT) on existing highway facilities may be obtained from highway inventory sheets. Table 1 may be used to find directional peak hour volumes. The average directional peak hour volumes on the new highway facilities (from step 3.A) may be added to parallel link segments on the old highway facility --

$$DPHV_{kl} = \underbrace{DPHV'_{kl} + DPHV'_{kol}}_{\text{from step 3.A}}$$

C. Using parameters from the VROAD speed-capacity correspondence Table 2, average travel speeds may be estimated:

$$S_{kl} = \frac{So}{1+0.15(V/C)^4} = \frac{So}{1+0.15\left(\frac{DPHV_{kl}}{C}\right)^4}$$

where:

So = mean free speed

C = "practical capacity:
= 0.75 x ("possible" capacity per lane)
x (number of lanes in each direction)

D. Daily peak period travel emissions prior to building the new highway facility may be estimated as:

$$Ep = 4\sum_{l}\left(\sum_{k} d_k \times (DPHV_{kl}) \times e(S_{kl})\right)$$

E. The net reduction in daily peak period emissions would then be

$$\begin{aligned} E &= Ep - Ep' \\ &= 4\sum_{l}\Big[\sum_{k} d_k \times (DPHV_{kl}) \times e(S_{kl}) - \sum_{k} d_k \times (DPHV'_{kl}) \times e(S'_{kl}) \\ &\qquad - \sum_{ko} d_{ko} \times (DPHV'_{kd}) \times e(S'_{kol})\Big] \end{aligned}$$

TABLE 1

PERCENTAGE OF AADT IN PEAK HOUR FOR ONE DIRECTION AND BOTH DIRECTIONS BY PEAK HOUR, 30TH HIGHEST HOUR, AND 200TH HIGHEST HOUR, BY TYPE OF FACILITY

TYPE OF FACILITY	PERCENTAGE OF AADT IN PEAK HOURS					
	ONE DIRECTION			BOTH DIRECTIONS		
	PEAK HOUR	30TH HIGHEST HOUR	200TH HIGHEST HOUR	PEAK HOUR	30TH HIGHEST HOUR	200TH HIGHEST HOUR
Rural:						
Freeway	23.6	15.4	11.4	18.3	13.5	10.9
Expressway	21.5	14.1	10.6	19.2	12.7	9.7
Highway with more than 2 lanes	21.2	13.7	10.3	16.4	12.7	9.9
2-Lane two-way highway	—	—	—	19.7	13.6	11.2
Urban:						
Freeway	15.0	12.7	10.7	13.6	11.0	9.6
Expressway	14.6	11.4	8.9	11.6	9.5	8.3
Street with more than 2 lanes	13.8	11.1	9.6	12.0	10.0	8.7
2-Lane two-way street	—	—	—	13.4	10.6	9.0

Source: 1965 Highway Capacity Manual, Table 3.12, p. 40.

TABLE 2

FLORISSANT 5-ITERATION ASSIGNMENT USING BASENET.CCI8

80CT80 21.40.21 URCAD REPCRT 10

LINK SPEED AND CAPACITY TABLES

AREA TYPE 1

LANES		FACILITY TYPE					
		1	2	3	4	5	6
1	CAPACITY	1750.	800.	600.	700.	10000.	600
	SPEED	45.0	35.0	20.0	22.0	10.0	22.0
2	CAPACITY	1750.	800.	600.	700.	10000.	600.
	SPEED	45.0	35.0	20.0	22.0	10.0	22.0
3	CAPACITY	1750.	800.	600.	700.	10000.	600.
	SPEED	45.0	35.0	20.0	22.0	10.0	22.0
4	CAPACITY	1750.	800.	600.	700.	10000.	600.
	SPEED	45.0	35.0	20.0	22.0	10.0	22.0
5	CAPACITY	1750.	800.	600.	700.	10000.	600.
	SPEED	45.0	35.0	20.0	22.0	10.0	22.0
6	CAPACITY	1750.	800.	600.	700.	10000.	600.
	SPEED	45.0	35.0	20.0	22.0	10.0	22.0
7	CAPACITY	1750.	800.	600.	700.	10000.	600.
	SPEED	45.0	35.0	20.0	22.0	10.0	22.0
8	CAPACITY	1750.	800.	600.	700.	10000.	600.
	SPEED	45.0	35.0	20.0	22.0	10.0	22.0
9	CAPACITY	1750.	800.	600.	700.	10000.	600.
	SPEED	45.0	35.0	20.0	22.0	10.0	22.0

AREA TYPE 2

LANES		FACILITY TYPE					
		1	2	3	4	5	6
1	CAPACITY	1750.	1000.	550.	550.	10000.	800.
	SPEED	45.0	40.0	25.0	29.0	15.0	25.0
2	CAPACITY	1750.	1000.	550.	550.	10000.	800.
	SPEED	45.0	40.0	25.0	29.0	15.0	25.0
3	CAPACITY	1750.	1000.	550.	550.	10000.	800.
	SPEED	45.0	40.0	25.0	29.0	15.0	25.0
4	CAPACITY	1750.	1000.	550.	550.	10000.	800.
	SPEED	45.0	40.0	25.0	29.0	15.0	25.0
5	CAPACITY	1750.	1000.	550.	550.	10000.	800.
	SPEED	45.0	40.0	25.0	29.0	15.0	25.0
6	CAPACITY	1750.	1000.	550.	550.	10000.	800.
	SPEED	45.0	40.0	25.0	29.0	15.0	25.0
7	CAPACITY	1750.	1000.	550.	550.	10000.	800.
	SPEED	45.0	40.0	25.0	29.0	15.0	25.0
8	CAPACITY	1750.	1000.	550.	550.	10000.	800.
	SPEED	45.0	40.0	25.0	29.0	15.0	25.0
9	CAPACITY	1750.	1000.	550.	550.	10000.	800.
	SPEED	45.0	40.0	25.0	29.0	15.0	25.0

UNITSO CAPACITY IN %VEH/LANE /HR
SPEED IN MI/HR

TABLE 2
(continued)

AREA TYPE 3

LANES		FACILITY TYPE 1	2	3	4	5	6
1	CAPACITY	1750.	1100.	550.	900.	10000.	800.
	SPEED	50.0	40.0	30.0	32.0	15.0	28.0
2	CAPACITY	1750.	1100.	550.	900.	10000.	800.
	SPEED	50.0	40.0	30.0	32.0	15.0	28.0
3	CAPACITY	1750.	1100.	550.	900.	10000.	800.
	SPEED	50.0	40.0	30.0	32.0	15.0	28.0
4	CAPACITY	1750.	1100.	550.	900.	10000.	800.
	SPEED	50.0	40.0	30.0	32.0	15.0	28.0
5	CAPACITY	1750.	1100.	550.	900.	10000.	800.
	SPEED	50.0	40.0	30.0	32.0	15.0	28.0
6	CAPACITY	1750.	1100.	550.	900.	10000.	800.
	SPEED	50.0	40.0	30.0	32.0	15.0	28.0
7	CAPACITY	1750.	1100.	550.	900.	10000.	800.
	SPEED	50.0	40.0	30.0	32.0	15.0	28.0
8	CAPACITY	1750.	1100.	550.	900.	10000.	800.
	SPEED	50.0	40.0	30.0	32.0	15.0	28.0
9	CAPACITY	1750.	1100.	550.	900.	10000.	800.
	SPEED	50.0	40.0	30.0	32.0	15.0	28.0

AREA TYPE 4

LANES		FACILITY TYPE 1	2	3	4	5	6
1	CAPACITY	1750.	1000.	550.	650.	10000.	800.
	SPEED	45.0	35.0	20.0	24.0	15.0	22.0
2	CAPACITY	1750.	1000.	550.	650.	10000.	800.
	SPEED	45.0	35.0	20.0	24.0	15.0	22.0
3	CAPACITY	1750.	1000.	550.	650.	10000.	800.
	SPEED	45.0	35.0	20.0	24.0	15.0	22.0
4	CAPACITY	1750.	1000.	550.	650.	10000.	800.
	SPEED	45.0	35.0	20.0	24.0	15.0	22.0
5	CAPACITY	1750.	1000.	550.	650.	10000.	800.
	SPEED	45.0	35.0	20.0	24.0	15.0	22.0
6	CAPACITY	1750.	1000.	550.	650.	10000.	800.
	SPEED	45.0	35.0	20.0	24.0	15.0	22.0
7	CAPACITY	1750.	1000.	550.	650.	10000.	800.
	SPEED	45.0	35.0	20.0	24.0	15.0	22.0
8	CAPACITY	1750.	1000.	550.	650.	10000.	800.
	SPEED	45.0	35.0	20.0	24.0	15.0	22.0
9	CAPACITY	1750.	1000.	550.	650.	10000.	800.
	SPEED	45.0	35.0	20.0	24.0	15.0	22.0

UNITS: CAPACITY IN VEH/LANE /HR
SPEED IN MI/HR

TABLE 2
(continued)

AREA TYPE 5

LANES		FACILITY TYPE					
		1	2	3	4	5	6
1	CAPACITY	1750.	1100.	550.	900.	10000.	800.
	SPEED	55.0	45.0	30.0	32.0	15.0	28.0
2	CAPACITY	1750.	1100.	550.	900.	10000.	800.
	SPEED	55.0	45.0	30.0	32.0	15.0	28.0
3	CAPACITY	1750.	1100.	550.	900.	10000.	800.
	SPEED	55.0	45.0	30.0	32.0	15.0	28.0
4	CAPACITY	1750.	1100.	550.	900.	10000.	800.
	SPEED	55.0	45.0	30.0	32.0	15.0	28.0
5	CAPACITY	1750.	1100.	550.	900.	10000.	800.
	SPEED	55.0	45.0	30.0	32.0	15.0	28.0
6	CAPACITY	1750.	1100.	550.	900.	10000.	800.
	SPEED	55.0	45.0	30.0	32.0	15.0	28.0
7	CAPACITY	1750.	1100.	550.	900.	10000.	800.
	SPEED	55.0	45.0	30.0	32.0	15.0	28.0
8	CAPACITY	1750.	1100.	550.	900.	10000.	800.
	SPEED	55.0	45.0	30.0	32.0	15.0	28.0
9	CAPACITY	1750.	1100.	550.	900.	10000.	800.
	SPEED	55.0	45.0	30.0	32.0	15.0	28.0

```
UNITSO CAPACITY IN %VEH/LANE /HR
       SPEED IN MI/HR

UROAD LLL) % INFORMATION 0 188K BYTES OF CORE NEEDED FOR NETWORK INPUT

ROA2 2600 % INFORMATION 0  OF ON OLDHR AT NODE 4040  858 LINKS READ
```

IX.

PROCEDURES FOR ANALYZING TRANSPORTATION CONTROL MEASURES INTERSECTION AND INTERCHANGE IMPROVEMENTS

GENERAL DISCUSSION

Improvements made to isolated intersections or interchanges will generally be reflected in a reduction in average vehicle delay and a concomitant decrease in air pollution emissions due to reduced vehicle idling. The Critical Movement Analysis technique may be used to estimate average delay per vehicle before and after improvements are made at a signalized intersection.[1] Since this technique requires a considerable degree of analysis for each application, it is suggested that (1) similar types of intersection improvements be categorized and (2) estimates be made of average delay per vehicle for one or two "typical" cases under each category. For the various categories of intersection or interchange improvements, it may be assumed that the "typical" change in average delay per vehicle entering an intersection may be applied to all intersections within a given category.

The steps to be followed are:

1. Estimate average delay per vehicle entering the intersection using the Critical Movement Analysis technique prior to the improvement.

2. Estimate average delay per vehicle entering the intersection using the Critical Movement Analysis technique after the improvement.

3. The reduction in emissions may be estimated as:

$$E_p = V_p \times \Delta D \times e$$

where:

E_p = reduction in emissions

V_p = peak travel period vehicles entering the intersection (assuming a 4-hour peak travel period)

$= 4 \times \sum_i (ADT_i) \times$ (fraction of ADT_i in peak hour from Table 1)

i = intersection approach

---

[1] Transportation Research Board, Interim Materials on Highway Capacity, Transportation Research Circular Number 212, January, 1980. Where detailed traffic engineering studies have been made, and delays estimated, then these values are likely to be more accurate.

where:

ADT_i = average daily traffic on highway i

ΔD = change in delay per vehicle, in hours per vehicle

e = emissions per vehicle-hour of idling

The remainder of this section consists of selected notes on the Critical Movement Analysis technique. Sections marked with an asterisk or underlined are those which are most applicable to the present assessment of TCM's in St. Louis.

TABLE 1

PERCENTAGE OF AADT IN PEAK HOUR FOR ONE DIRECTION AND BOTH DIRECTIONS BY PEAK HOUR, 30TH HIGHEST HOUR, AND 200TH HIGHEST HOUR, BY TYPE OF FACILITY

TYPE OF FACILITY	PERCENTAGE OF AADT IN PEAK HOURS					
	ONE DIRECTION			BOTH DIRECTIONS		
	PEAK HOUR	30TH HIGHEST HOUR	200TH HIGHEST HOUR	PEAK HOUR	30TH HIGHEST HOUR	200TH HIGHEST HOUR
Rural:						
Freeway	23.6	15.4	11.4	18.3	13.5	10.9
Expressway	21.5	14.1	10.6	19.2	12.7	9.7
Highway with more than 2 lanes	21.2	13.7	10.3	16.4	12.7	9.9
2-Lane two-way highway	—	—	—	19.7	13.6	11.2
Urban:						
Freeway	15.0	12.7	10.7	13.6	11.0	9.6
Expressway	14.6	11.4	8.9	11.6	9.5	8.3
Street with more than 2 lanes	13.8	11.1	9.6	12.0	10.0	8.7
2-Lane two-way street	—	—	—	13.4	10.6	9.0

Source: 1965 Highway Capacity Manual, Table 3.12, p. 40.

Critical Movement Analysis

DISCUSSION

Introduction

Critical Movement Analysis is a procedure which allows for capacity and level of service determination for signalized intersections. The analysis incorporates the effects of geometry and traffic signal operation and results in a level of service determination for the intersection as a whole operating unit.

The ability of a line of vehicles to discharge past a point is the key principle involved. Rarely can a discharge rate of 2000 passenger cars per hour of green be surpassed. Because of time lost due to queue start up and signal change intervals the maximum discharge of a single lane at signalized intersections typically varies from 1500 to 1800 passenger cars per hour of green. The 1965 Highway Capacity Manual (HCM) (1) states that a single lane at a traffic signal can accommodate 2000 and 1500 passenger cars per hour of green respectively, for a perfectly coordinated signal where all vehicles pass through without stopping, and for a signal where all vehicles must stop.

Definitions

*Approach - The portion of an intersection leg which is used by traffic approaching the intersection.

*Capacity - The maximum number of vehicles that has a reasonable expectation of passing over a given roadway or section of roadway in one direction during a given time period under prevailing roadway and traffic conditions.

*Change Interval - Yellow time plus all red time occurring between two phases.

*Critical Volume - A volume (or combination of volumes) for a given street which produces the greatest utilization of capacity (e.g., needs the greatest green time) for that street. Given in terms of passenger cars or mixed vehicles per hour per lane.

*Cycle Time - The period in seconds required for one complete sequence of signal indications.

*Delay - Stopped time delay per approach vehicle, in seconds per vehicle.

*Green Time - The length of a green phase plus its change interval, in seconds.

*Hourly Volume - The number of (mixed) vehicles that pass over a given section of a lane or roadway during a time period of one hour.

*Level of Service - A measure of the mobility characteristics of an intersection, as determined by vehicle delay and a secondary factor, volume/capacity ratio.

*Passenger Car Equivalency - For a given vehicle, the number of through moving passenger cars it is equivalent to, based on its headway and delay creating effects.

*Passenger Car Volumes - The volumes expressed in terms of passenger cars, following the application of passenger car equivalency factors to vehicular volumes.

*Phase - A part of the cycle allocated to any traffic movement or combination of traffic movements receiving right of way simultaneously during one or more intervals.

*Probable Phase - A phase within the probable sequence of phases which represents the sequence of a multi-phase signal controller most likely to occur under given traffic conditions.

*LOS = Level of Service
*LT = Left Turn
*PCE = Passenger Car Equivalency
*pch = Passenger cars per hour
*PCV = Passenger Car Volume, in pch

*G/C = Green time/Cycle time ratio
*TH = Through Traffic
*U = Lane Utilization Factor
*v/c = Volume/Capacity ratio
*V_L = Left Turn Volume, in vph
*V_0 = Volume Opposing a V_L, in vph
*vph = Vehicles per hour (mixed traffic)
*RT = Right Turn

*Left Turns. Left turning vehicles are treated in considerable detail in most capacity computation techniques. The reason for this is simple--left turns (unless removed from through traffic lanes by provision of exclusive turn lanes) have a large impact on capacity and on vehicular delay, which will be the principal determinant of level of service in the new HCM.

The most direct means of taking into account the delaying effects of left turn vehicles is to convert them to pch using PCE values. It is anticipated that future research will lead to a range of PCE values for various combinations of geometry, traffic volumes, opposing traffic volumes, and signal phasing for left turns.

Table 3 gives the PCE values for left turns for use when applying the critical movement procedure. These values are to be considered as "average" values for a broad range of traffic and geometric conditions. Future research may lead to a more precise formulation of left turn PCE values by incorporating other variables, in addition to "opposing traffic."

✱Table 3. PCE Values: Left Turn Effects

Left Turns Allowed from Left-Through Lanes[a]					
1. No Turn Phase	Opposing Volume, in vph: 1 left turn equals:	0-299 1.0 PCE	300-599 2.0 PCE	600-999 4.0 PCE	1000 + 6.0 PCE
2. With Turn Phase	1 left turn equals 1.2 PCE				

[a] PCE Values are used in Step 5, PLANNING applications, to develop a distribution of volumes among several traffic lanes.

✱*Lane Utilization*

Critical Movement Analysis is based on "per lane" volumes. Thus, for movements (e.g., left turn, through, and right turn) which take place from more than one lane, it is necessary to estimate the volume in each of the lanes affected. In this manner, the highest lane volume can be identified and used in the analysis.

Reilly and Bellis (11, 12, 13) indicate that a traffic movement carried in two lanes could break down into a 55% / 45% split, by lane. A traffic movement carried in three lanes might divide into a 40% / 35% / 25% split.

In the critical movement analysis proposed by Messer and Fambro (5) a lane utilization factor is applied. For two lanes, a 55% / 45% split in volume is assumed. For three lanes, 40% of the total movement is assumed to occur in the most heavily used lane. Many HCM users have used analyses based on the assumption that volume is distributed approximately equally by lane, especially under peak conditions.

✱*Levels of Service*

As part of the critical movement technique, a set of guidelines on volume/capacity (v/c) ratio, average delay values, and sum of critical volumes is presented for use, review, and comment by HCM users. Table 6 gives the recommended thresholds for the sum of critical volumes for Levels of Service A through E for both the PLANNING and the OPERATIONS AND DESIGN applications.

Table 6. Level of Service Ranges

✱PLANNING Applications (in vph)

Level of Service	Maximum Sum of Critical Volumes Two Phase	 Three Phase	 Four or more Phases
A	900	855	825
B	1050	1000	965
C	1200	1140	1100
D	1350	1275	1375
E	1500	1425	1225
F	---------not applicable---------		

✱*Delay*

Because delay will be the principal determinant of signalized intersection level of service in the new HCM, Table 7 is included. The delay values given are not yet an integral part of the Critical Movement Analysis procedure but are presented as an initial step in developing a range of delay values which can be related to intersection level of service. The values of Table 7 do not take into account the offset relationship between adjacent signals. Synthesis of data from a number of sources has been used to produce Table 7. HCM users may find it useful to compare the table with locally obtained delay data.

✱Table 7. Delay and Level of Service

Level of Service	Typical v/c Ratio	Delay Range[a] (secs. per veh.)
A	0.00-0.60	0.0-16.0
B	0.61-0.70	16.1-22.0
C	0.71-0.80	22.1-28.0
D	0.81-0.90	28.1-35.0
E	0.91-1.00	35.1-40.0
F	varies	40.1 or greater

[a] Measured as "stopped delay" as described in Ref. (17). Delay values relate to the mean stopped delay incurred by all vehicles entering the intersection. Note that traffic signal coordination effects are not considered and could drastically alter the delay range for a given v/c ratio.

Source: W. R. Reilly (NCHRP Project 3-28), based on a synthesis of various data.

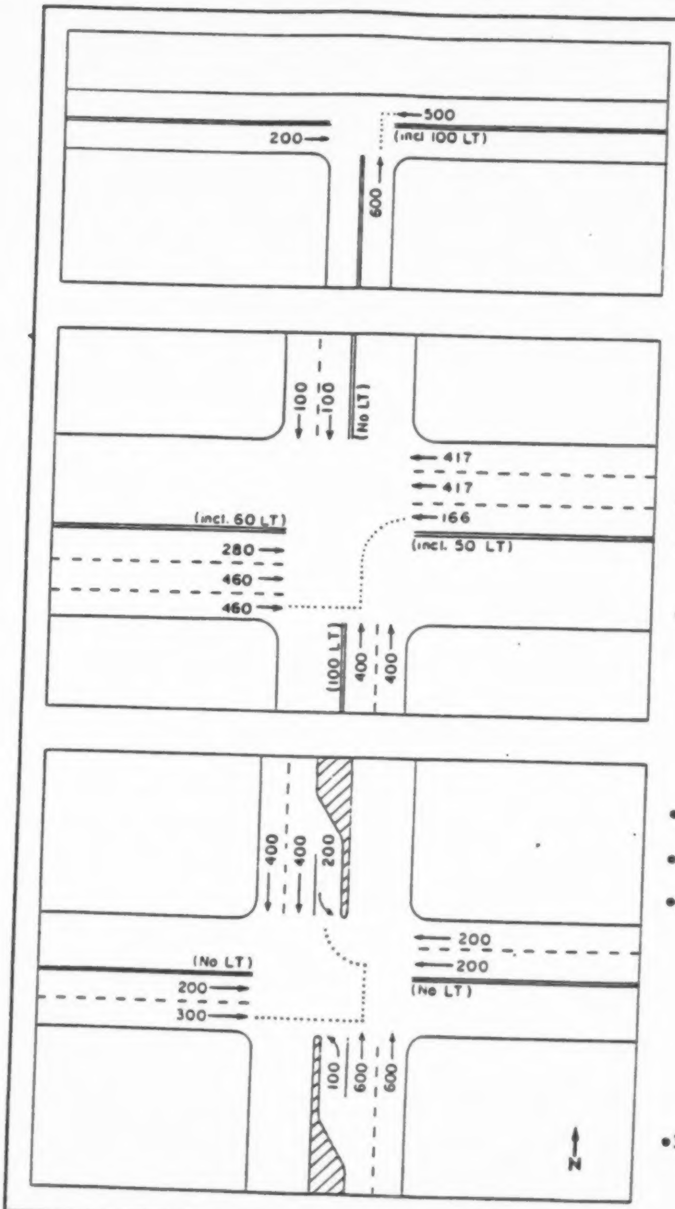
✱*PLANNING Applications*

In these applications, an important reference work is that of McInerney and Petersen (3). The only tabular material used is that found in Table 6 which gives a single value for the maximum sum of critical lane volumes, in vehicles per hour, assuming "average" traffic, signal, and geometric conditions, and Table 3, which is used to apportion traffic among several lanes. + TABLE 7 (DELAYS)

The focus of this tool is to allow for a rapid approximation of level of service. None of the detailed individual adjustment factors need be applied to obtain a solution. The solution is for typical average conditions and should not necessarily be used for detailed design or operational decisions.

Critical Movement Analysis

* Figure 1. Critical Movements, PLANNING Applications



- Single Lane Approaches
- Two Phase Signal
- Critical Movements:
  500 and 600 vph
- Sum of Critical Volumes:
  500 + 600 = 1100 vph

- Two and Three Lane Approaches
- Two Phase Signal
- Critical Movements, by lane:
  400 and 460 + 50 vph
- Sum of Critical Volumes:
  400 + 510 = 910 vph

- Two and Three Lane Approaches
- Five Phase Actuated Signal
- Note:
  For the east-west street, the critical volume is 300 vph. For the north-south street the greatest demand for green time will occur with the conflicting movement totaling 800 vph (600 + 200 LT). The conflicting movement totaling 500 vph (400 + 100 LT) would require less green time and will be satisfied if the 800 vph critical volume is satisfied.
- Sum of Critical Volumes:
  300 + 800 = 1100 vph.

Note:
The above examples relate to PLANNING applications of Critical Movement Analysis.

USER APPLICATIONS

Methodology

The intent of this section is to set forth the detailed procedures, with example problems, to be used in Critical Movement Analysis. The examples are divided into two groups: PLANNING applications with quick and simple solutions, and OPERATIONS AND DESIGN applications with more complex detailed solutions. A Calculation Form has been developed for each of the two groups of applications. These forms are shown in the following pages. Detailed definitions, the analytical framework, and references used in Critical Movement Analysis, are described in the preceeding section entitled "DISCUSSION."

PLANNING applications are carried out in terms of mixed vehicles per hour (vph).

Definitions

The abbreviations and symbols used in critical movement analysis are defined below. A more detailed set of definitions of concepts and terms is found in the preceeding "DISCUSSION".

G/C = Green time/Cycle time ratio
HV = Hourly Volume
LB = Local Bus (Number per hour)
LOS = Level of Service
LT = Left Turn
PCE = Passenger Car Equivalency
pch = Passenger cars per hour
PCV = Passenger Car Volume, in pch
PHF = Peak Hour Factor
PV = Period Volume
RT = Right Turn
T = Truck and Through Bus (Percentage of HV)
TH = Through Traffic
U = Lane Utilization Factor
v/c = Volume/Capacity ratio
V_L = Left Turn Volume, in vph
V_0 = Volume Opposing a V_L, in vph
vph = Vehicles per hour (mixed traffic)
W = Lane Width factor

PLANNING Applications: Procedure

The PLANNING application of Critical Movement Analysis is based on average or better conditions of geometry and traffic. The solutions can resolve the following questions:

1. What is the operating level of service for a signalized intersection as a whole?

2. If a design level of service is set, what changes in lane geometry or demand volume will be necessary to achieve that level?

3. What changes in lane configuration or signal phasing will have the greatest impact on operating level of service?

Step-By-Step Approach

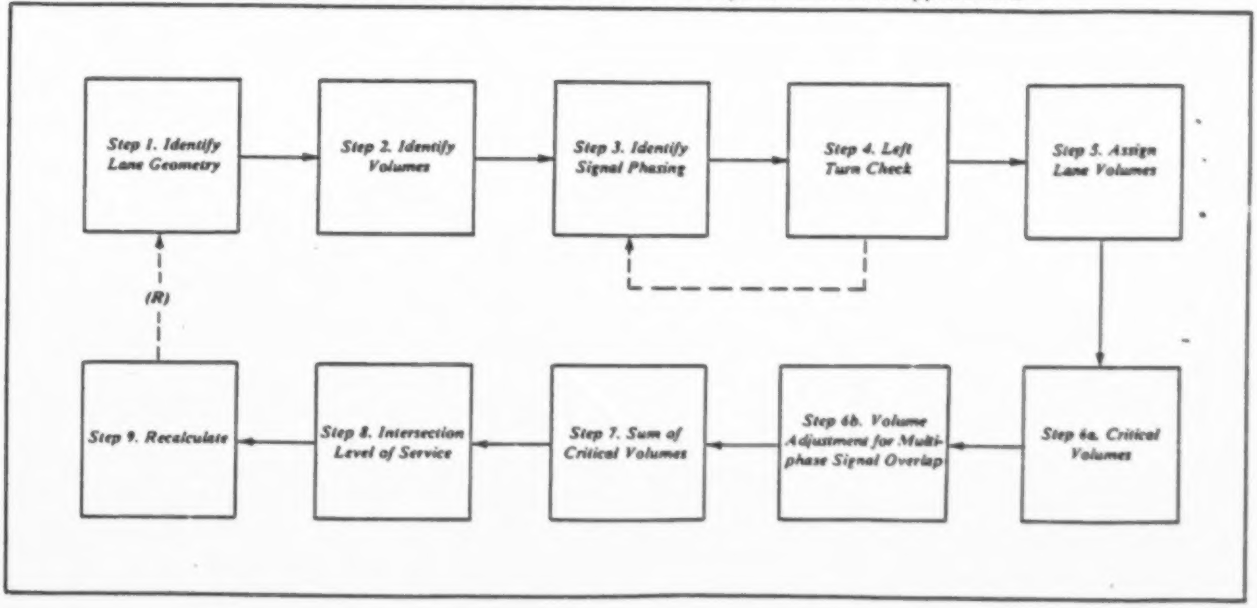
The steps followed in solving a problem by this technique are described below. Figure 2 contains an illustration of the steps followed, which are:

Step 1. Identify Lane Geometry - the assumed or known lane configuration for each approach is identified, by type of lane.

Step 2. Identify Volumes - the assumed or known traffic volumes for the design hour or analysis hour are identified in vehicles per hour. Left turn volumes, through, and right turn volumes are identified for each intersection approach.

Step 3. Identify Phasing - the signal phasing to be used for analysis is identified.

Figure 2. Procedure for Critical Movement Analysis, PLANNING Applications


Step 1. Identify Lane Geometry
Step 2. Identify Volumes
Step 3. Identify Signal Phasing
Step 4. Left Turn Check
Step 5. Assign Lane Volumes
(R)
Step 9. Recalculate
Step 8. Intersection Level of Service
Step 7. Sum of Critical Volumes
Step 6b. Volume Adjustment for Multi-phase Signal Overlap
Step 6a. Critical Volumes

Step 4. Left Turn Check - for an assumed phasing with no left turn phases, a check is made on the probability of clearing the identified left turn volume. On the change interval, 2.0 times the number of cycles per hour gives the maximum number of lefts that can clear on the change interval. Use 90 left turns per hour if no information on number of cycles per hour is available. Additionally, the number of vehicles per hour that can clear through opposing traffic during the green interval is estimated by:

$$V_L = (G/C)(1200) - V_0$$

where:

V_L = Left Turn Volume, in vph, that can clear through opposing traffic on the green interval

G/C = Green time/Cycle time ratio for opposing flow (V_0). If no other design information is available, estimate by lane volume ratio.

V_0 = Volume of Opposing through plus right turn traffic, in vph.

Note that the green time in the G/C ratio is considered as the green interval plus the change interval. If the sum of the two left turn volumes described above is less than the analysis volume, a separate left turn phase can be considered, by returning to Step 3. If the sum is greater than the left turn analysis volume, no special left turn phasing needs to be considered and the analysis moves to Step 5.

The purpose of the left turn check is to determine whether all left turn movements not controlled by an exclusive turn phase can be accommodated. If not, the assumption on signal phasing can be changed to provide for left turn phasing. In many cases (e.g., analysis of existing conditions), no change in phasing is assumed and the analysis continues, with the analyst knowing that the non-satisfied left turns will create operating difficulties and be subject to excessive delay.

Step 5. Assign Lane Volumes - the volumes are assigned to the appropriate lanes. If no left turn lanes exist, the left turn volume is converted to a pch volume (Table 3) and the remaining through plus right turn volume is assumed to be in pch units. The The sum of these two pch volumes is then divided equally among all approach lanes. However, in all cases, the entire left turn volume must be assigned to the lane(s) from which the turns are made, and the remaining pch volume for through and right turn traffic is distributed equally among the remaining lanes. Following this distribution, the pch volume is converted back to vehicles per hour for the lane carrying the left turn.

If a left turn lane exists, the left turn volume in vehicles per hour is assigned to that lane and the through plus right turn volume is divided equally among the through and through-right lanes. For the special case of a double left turn lane, fifty-five percent of the total left turn volume is assigned to one left turn lane and forty-five percent to the other.

Step 6. Critical Volumes - for each signal phase, the highest total of conflicting traffic (on a per lane basis) is identified. For a two phase signal, the "highest total of the through (or through plus right turn if no exclusive right turn lane exists) plus the opposing left turn volume" is selected. For a three-to-eight phase ("multiphase") signal, each phase listed in the typical (i.e., most probable) phase sequence has one critical volume. The most probable phase sequence represents the sequence of a multiphase signal most likely to occur under the volume conditions assigned in Step 5. Where an exclusive right turn lane exists, such a lane is often not included in the critical analysis if right turns on red are permitted. However, such a lane can be included if the analyst believes that it might carry the most critical volume for that approach. Some reduction (30 percent is typical) in the assigned right turn volume (Step 5) may be made to allow for right turns made on red. If right turns on red are not permitted, an exclusive right turn lane is included in the analysis. Note that Calculation Form 1 contains Step 6a, which is used for two phase signals, and Step 6b, which is used for multiphase signals. In Steps 6a and 6b, a street operating without separate turn phases must have the opposing left turns added to the through volume to obtain the critical volume for that street.

Step 7. Sum of Critical Volumes - the critical volumes, for each phase, are summed.

Step 8. Intersection Level of Service - the sum of critical volumes is compared with Table 6, and an intersection level of service is identified.

Step 9. Recalculate - depending on the solution found in Step 8, a change in geometry, demand volume, or signal phasing can be made, and a recalculation —Steps1(R) through 9(R)—is performed.

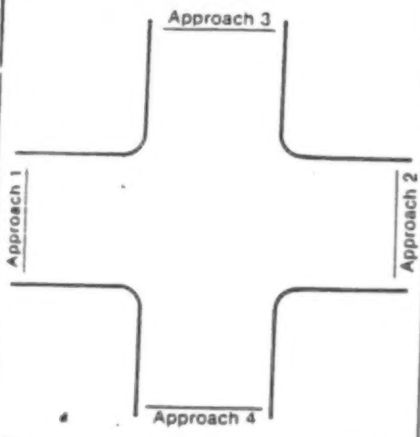
Calculation Form 1 is used for PLANNING applications.

Critical Movement Analysis: PLANNING
Calculation Form 1

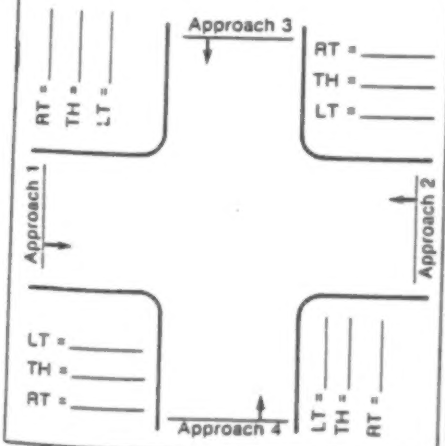
Intersection ______________________ **Design Hour** __________

Problem Statement ______________________

Step 1. Identify Lane Geometry



Step 2. Identify Volumes, in vph



Step 3. Identify Phasing

☐ ________

☐ ________

☐ ________

☐ ________

☐ ________

☐ ________

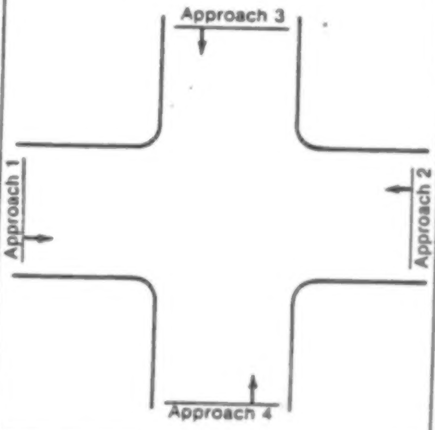
A1 → A3 ↓ B1 ┌ B3 ┐

A2 ← A4 ↑ B2 ┘ B4 └

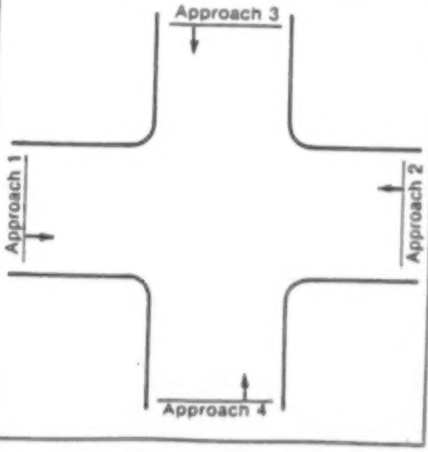
Step 4. Left Turn Check

	Approach 1	2	3	4
a. Number of change intervals per hour				
b. Left turn capacity on change interval, in vph				
c. G/C Ratio				
d. Opposing volume in vph				
e. Left turn capacity on green, in vph				
f. Left turn capacity in vph (b + e)				
g. Left turn volume in vph				
h. Is volume > capacity (g > f)?				

Step 5. Assign Lane Volumes, in vph



Step 6a. Critical Volumes, in vph (two phase signal)



Step 6b. Volume Adjustment for Multiphase Signal Overlap

Probable Phase	Possible Critical Volume in vph	Volume Carryover to next phase	Adjusted Critical Volume in vph

Step 7. Sum of Critical Volumes

______ + ______ + ______ + ______

= ______ vph

Step 8. Intersection Level of Service

(compare Step 7 with Table 6)

☐

Step 9. Recalculate

Geometric Change ______________

Signal Change ______________

Volume Change ______________

Comments

PLANNING Applications: Example 1

Problem

Lane configuration and peak hour volumes are shown on Calculation Form 1 for an existing urban intersection. The following three questions must be answered:

1. What is the intersection level of service?
2. Can left turns be handled without installing an exclusive phase?
3. If left turn lanes are added on Approaches 3 and 4 what changes, if any, may be expected in the level of service?

Analysis

Step 1. Identify Lane Geometry. Existing lane configuration is shown on Calculation Form 1.

Step 2. Identify Volumes. Existing peak hour volumes (vph) are shown on Calculation Form 1. Approaches are numbered 1, 2, 3, and 4, from the west, east, north, and south, respectively.

Step 3. Identify Phasing. A two phase signal operation exists.

Step 4. Left Turn Check. A 90 second peak hour cycle length is used. Forty cycles per hour times 2.0 left turns per cycle result in 80 left turns per hour made on the change interval. Additionally, left turns made through opposing traffic on the green interval, assuming a 0.55 G/C ratio for Approaches 1 and 2 and a 0.45 G/C ratio for Approaches 3 and 4 are calculated by the formula:

$$V_L = (G/C)(1200) - V_0.$$

For all directions, the capacity for left turns is equal to or greater than left turn demand. Therefore, the two phase signal operation is adequate. Note that for left turns from Approach 3, demand and capacity are equal at 90 vph.

Step 5. Assign Lane Volumes. For Approaches 1 and 2, left turn volumes are assigned to the left turn lanes and through plus right turn volumes are divided equally between the remaining lanes.

For Approaches 3 and 4, factors from Table 3 are used to convert 90 and 120 left turns (with 530 vph and 330 vph opposing, respectively) to 180 and 240 pch, respectively. Thus, a total pch volume of 510 (from Approach 3) and 770 (from Approach 4) is computed. On a per lane basis, 255 pch and 385 pch, from Approaches 3 and 4, respectively, are computed.

For Approach 3, the left lane is assigned 255 pch, of which 180 pch is due to left turn vehicles. The right lane is also assigned 255 pch, comprised of through and right turn traffic. Therefore, the left lane carries 165 vph (90 left turns plus the difference between 180 and 255) and the right lane carries 255 vph.

For Approach 4, the left lane is assigned 385 pch, of which 240 pch are due to left turn vehicles. The right lane is also assigned 385 pch, comprised of through and right turn traffic. Thus, the left lane carries 265 vph (120 left turns plus the difference between 240 and 385) and the right lane carries 385 vph.

Table 6. Level of Service Ranges

	PLANNING Applications (in vph)		
	Maximum Sum of Critical Volumes		
Level of Service	Two Phase	Three Phase	Four or more Phases
A	900	855	825
B	1050	1000	965
C	1200	1140	1100
D	1350	1275	1375
E	1500	1425	1225
F	---------not applicable---------		

OPERATIONS AND DESIGN Applications (in pch)

(deleted)

The per lane volumes are entered in Step 5 of Calculation Form 1.

Step 6. Critical Volumes. Critical volumes for phase A1A2, on Approaches 1 and 2, is 795 + 40 LT or 455 + 50 LT. Use 835. Critical Volumes for phase A3A4 on Approaches 3 and 4 is 255 + 120 LT or 385 + 90 LT. Use 475. These volumes are graphically shown in Step 6A on the form.

Step 7. Sum of Critical Volumes. The sum of critical volumes is 835 + 475 or 1310 vph.

Step 8. Intersection Level of Service. Using Table 6, this value falls within the range of 1201 to 1350 vph or Level of Service D for two phase signals. The left turns can be handled using the geometry shown and a two phase signal.

Step 9. Recalculate. To determine the effect on level of service of adding left turn lanes on Approaches 3 and 4, return to Step 1 and recompute..

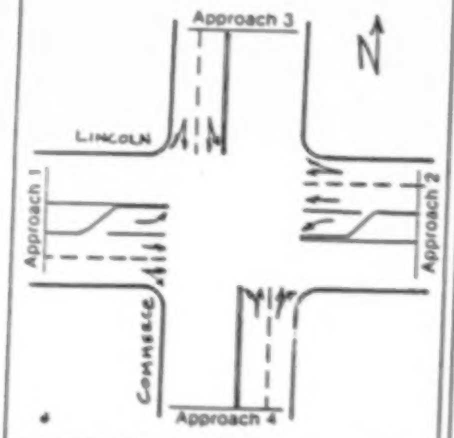
Critical Movement Analysis: PLANNING
Calculation Form 1

Example 1

Intersection LINCOLN AND COMMERCE **Design Hour** 4:30 - 5:30 p.m.

Problem Statement FIND EXISTING LOS. CAN LT BE HANDLED WITH 2 ϕ ?

Step 1. Identify Lane Geometry



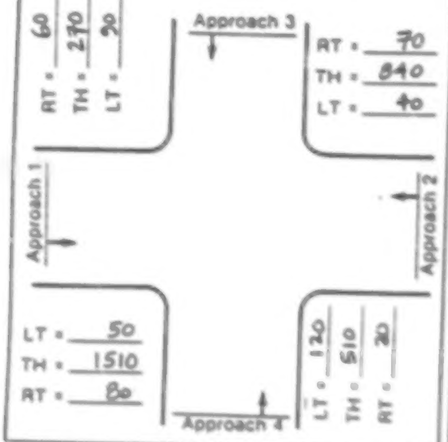
Step 4. Left Turn Check

	Approach 1	Approach 2	Approach 3	Approach 4
a. Number of change intervals per hour	40	40	40	40
b. Left turn capacity on change interval, in vph	80	80	80	80
c. G/C Ratio	.55	.55	.45	.45
d. Opposing volume in vph	910	1590	530	330
e. Left turn capacity on green, in vph	0	0	10	210
f. Left turn capacity in vph (b + e)	80	80	90	290
g. Left turn volume in vph	50	40	90	120
h. Is volume > capacity (g > f)?	No	No	No*	No

Step 6b. Volume Adjustment for Multiphase Signal Overlap

Probable Phase	Possible Critical Volume in vph	Volume Carryover to next phase	Adjusted Critical Volume in vph
2ϕ			

Step 2. Identify Volumes, in vph



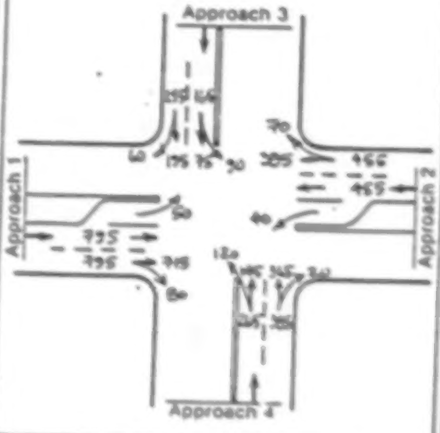
Approach 3: RT = 60 TH = 270 LT = 90

Approach 2: RT = 70 TH = 840 LT = 40

Approach 1: LT = 50 TH = 1510 RT = 80

Approach 4: LT = 120 TH = 510 RT = 20

Step 5. Assign Lane Volumes, in vph



Step 7. Sum of Critical Volumes

795 + 40 + 385 + 90

= 1310 vph

Step 8. Intersection Level of Service

(compare Step 7 with Table 6)

D

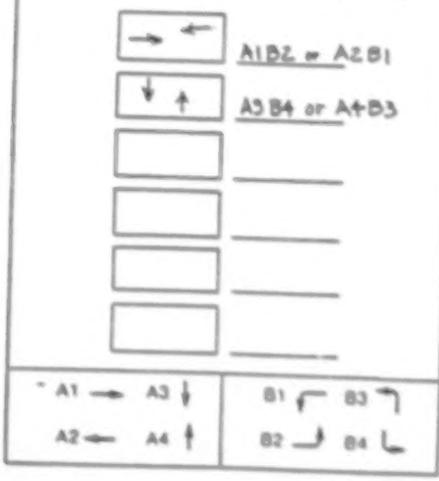
Step 9. Recalculate

Geometric Change ADD LT LANES TO APPROACHES 3 AND 4

Signal Change

Volume Change

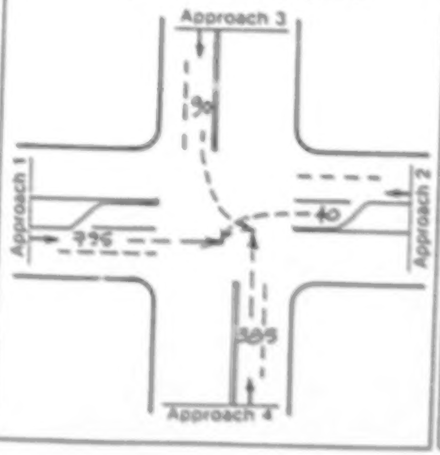
Step 3. Identify Phasing 2ϕ



A1B2 or A2B1

A3B4 or A4B3

Step 6a. Critical Volumes, in vph (two phase signal)



Comments

NOTE THAT LEFT TURN DEMAND

FOR APPROACH 3 EQUALS

CAPACITY

(Example 1)

Note: "(R)" denotes a recalculation.

Step 1(R). Identify Lane Geometry. Left turn lanes are added on Approaches 3 and 4.

Step 2(R). Identify Volumes. Volumes, in vph are shown on the form.

Step 3(R). Identify Phasing. The existing two phase signal will be analyzed.

Step 4(R). Left Turn Check. Step 4(R) is identical to the preceeding Step 5.

Step 5(R). Assign Lane Volumes. Left turns are assigned to left turn lanes and through plus right turn volumes are distributed equally to the remaining lanes.

Step 6(R). Critical Volumes. Critical volumes for phase A1A2 on Approaches 1 and 2 are 795 + 40 LT or 455 + 50 LT. Use 835. Critical volumes for phase A3A4 on Approaches 3 and 4 are 165 + 120 LT or 265 + 90 LT. Use 355.

Step 7(R). Sum of Critical Volumes. The sum of the critical volumes is (835 + 355) or 1190 vph.

Table 6. Level of Service Ranges

PLANNING Applications (in vph)

Level of Service	Maximum Sum of Critical Volumes		
	Two Phase	Three Phase	Four or more Phases
A	900	855	825
B	1050	1000	965
[✓ C]	1200	1140	1100
D	1350	1275	1375
E	1500	1425	1225
F	---------not applicable---------		

OPERATIONS AND DESIGN Applications (in pch)

(deleted)

Step 8(R). Intersection Level of Service. Using Table 6, the value of 1190 vph falls within the range of 1051 to 1200, or Level of Service C for two phase operation.

Step 9(R). Recalculate. No recalculation is necessary as it is demonstrated that left turn lanes alter the intersection Level of Service D to C.

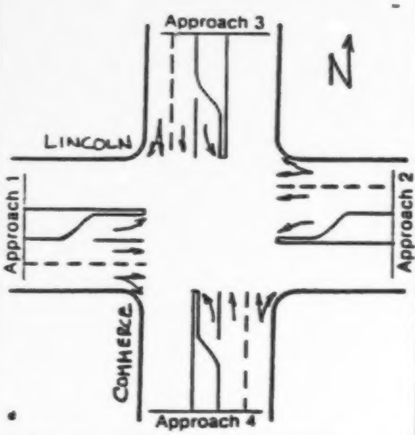
Critical Movement Analysis: PLANNING
Calculation Form 1

Example 1
(Recalculation)

Intersection LINCOLN AND COMMERCE **Design Hour** 4:30 - 5:30 p.m.

Problem Statement FIND CHANGE IN LOS BY ADDING LEFT-TURN LANES

Step 1. Identify Lane Geometry



Step 4. Left Turn Check

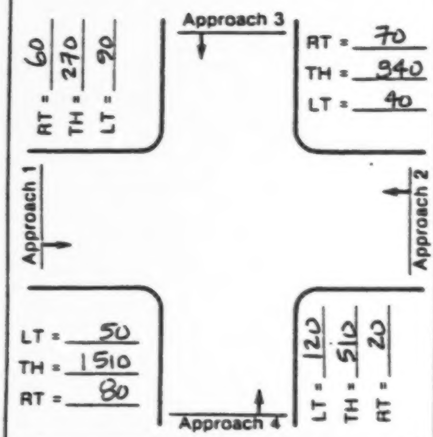
	Approach 1	2	3	4
a. Number of change intervals per hour	40	40	40	40
b. Left turn capacity on change interval, in vph	80	80	80	80
c. G/C Ratio	.55	.55	.45	.45
d. Opposing volume in vph	910	1590	530	330
e. Left turn capacity on green, in vph	0	0	10	210
f. Left turn capacity in vph (b + e)	80	80	90	290
g. Left turn volume in vph	50	40	90	120
h. Is volume > capacity (g > f)?	No	No	No*	No

Step 6b. Volume Adjustment for Multiphase Signal Overlap

Probable Phase	Possible Critical Volume in vph	Volume Carryover to next phase	Adjusted Critical Volume in vph

2ϕ

Step 2. Identify Volumes, in vph



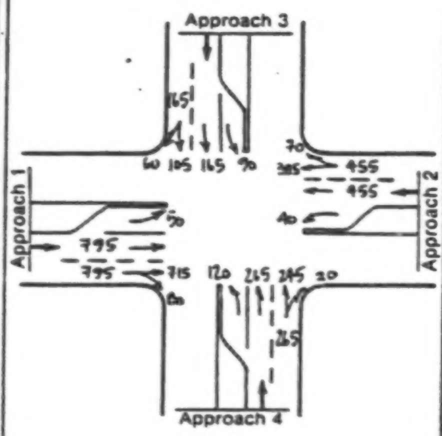
Approach 3: RT = 60, TH = 270, LT = 90

Approach 2: RT = 70, TH = 940, LT = 40

Approach 1: LT = 50, TH = 1510, RT = 80

Approach 4: LT = 120, TH = 510, RT = 20

Step 5. Assign Lane Volumes, in vph



Step 7. Sum of Critical Volumes

795 + 40 + 265 + 90

= 1190 vph

Step 8. Intersection Level of Service

(compare Step 7 with Table 6)

C

Step 9. Recalculate

NOT NECESSARY

Geometric Change ____

Signal Change ____

Volume Change ____

Step 3. Identify Phasing 2ϕ

→ ← A1B2 or A2B1

↓ ↑ A3B4 or A4B3

☐ ____

☐ ____

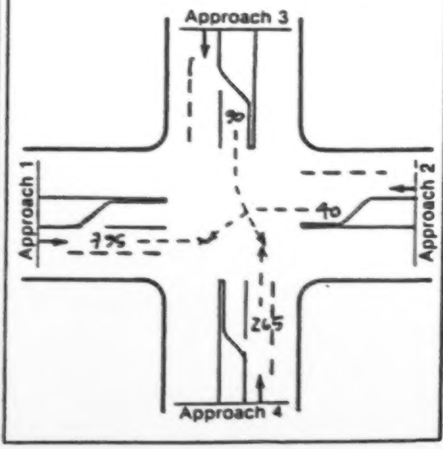
☐ ____

☐ ____

☐ ____

A1 → A3 ↓	B1 ↱ B3 ↰
A2 ← A4 ↑	B2 ↗ B4 ↳

Step 6a. Critical Volumes, in vph (two phase signal)



Comments

*NOTE THAT LEFT TURN DEMAND FOR APPROACH 3 EQUALS CAPACITY.

PLANNING Applications: Example 2

Problem

Lane configuration and design hour volumes (with left turn lanes on all approaches) are shown on the calculation form for a major new suburban intersection. The following information is needed.

1. The whole intersection level of service if an eight phase signal operation is used.

2. Change in level of service if an additional through lane is added to Approaches 3 and 4, and a right turn lane to Approaches 1 and 2.

Analysis

Step 1. Identify Lane Geometry. The assumed lane configuration is shown on the form.

Step 2. Identify Traffic Volumes. Design hour volumes are shown on the form.

Step 3. Identify Phasing. An eight phase signal is planned, with left turn arrows for each direction. The left turns are allowed only on the arrow (in a protected mode).

Step 4. Left Turn Check. Each left turn movement has a protected phase. Therefore, the left turn check is not needed.

Step 5. Assign Lane Volumes. Left turns are assigned to left turn lanes and through plus right turn volumes are distributed equally to the remaining lanes.

Step 6. Critical Volumes. Using Step 3, the phase sequence which most likely will appear under the volumes of Step 5 is: B2B1, A2B1, A1A2, B4B3, A4B3, and A3A4. For example, since left turn volume from Approach 2 (B1) is greater than left turn volume from Approach 1 (B2), B1 will continue receiving a green arrow after B2 has been terminated. Thus, A2B1 is selected as the most probable phase, rather than A1B2.

Table 6. Level of Service Ranges

PLANNING Applications (in vph)

Level of Service	Maximum Sum of Critical Volumes		
	Two Phase	Three Phase	Four or more Phases
A	900	855	825
B	1050	1000	965
C	1200	1140	1100
D	1350	1275	1375
Ⓔ	1500	1425	1225
F	---------not applicable---------		

OPERATIONS AND DESIGN Applications (in pch)

(deleted)

Using the most probable phase sequence, the through plus right turn volume which moves during the concurrent display of a left arrow is subtracted from the total through plus right turn volume and the remaining volume is carried over to the next phase. This calculation is listed in Step 6b on the form.

Step 7. Sum of Critical Volumes. The sum of critical lane volumes for all phases is 120 + 160 + 730 + 200 + 60 + 340, or 1610 vph.

Step 8. Intersection Level of Service. Using Table 6, the critical sum of 1610 vph falls beyond Level of Service E (1375 vph) for eight phase control. Therefore, the intersection will not operate without unacceptable delays.

Step 9. Recalculate. Return to Step 1 and recalculate to determine the effects of adding a through lane on Approaches 3 and 4, and a right turn lane on Approaches 1 and 2.

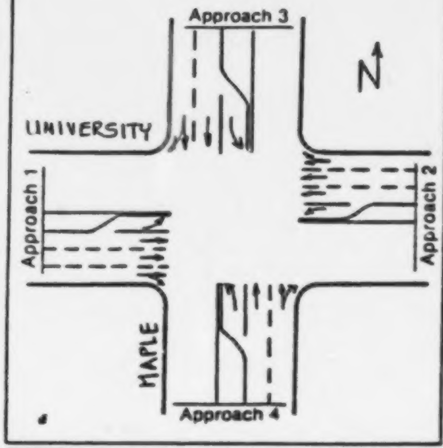
Critical Movement Analysis: PLANNING
Calculation Form 1

Example 2

Intersection UNIVERSITY AND MAPLE **Design Hour** 4:30 - 5:30 p.m.

Problem Statement FIND EXISTING LOS

Step 1. Identify Lane Geometry



Step 4. Left Turn Check

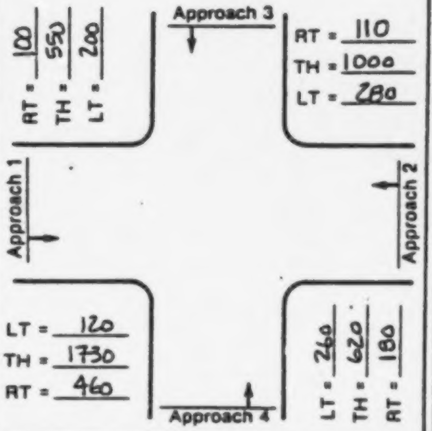
	Approach 1	2	3	4
a. Number of change intervals per hour		8φ		
b. Left turn capacity on change interval, in vph				
c. G/C Ratio				
d. Opposing volume in vph				
e. Left turn capacity on green, in vph				
f. Left turn capacity in vph (b + e)				
g. Left turn volume in vph				
h. Is volume > capacity (g > f)?				

Step 6b. Volume Adjustment for Multiphase Signal Overlap

Probable Phase	Possible Critical Volume in vph	Volume Carryover to next phase	Adjusted Critical Volume in vph
B2B1	120(B2)	280-120=160(B1)	120
A2B1	160 (B1)	370-160=210(A2)	160
A1A2	730(A1) OR 210(A2)		730
B4B3	200(B4)	260-200=60(B3)	200
A4B3	60(B3)	400-60=340(A4)	60
A3B4	325(A3) OR 340 (A4)		340

LT from 3
LT from 4

Step 2. Identify Volumes, in vph



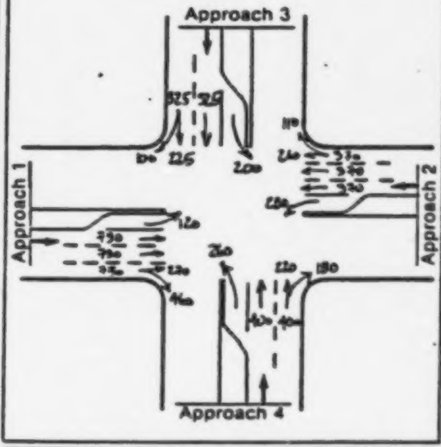
RT = 100
TH = 550
LT = 200

RT = 110
TH = 1000
LT = 280

LT = 120
TH = 1730
RT = 460

LT = 260
TH = 620
RT = 180

Step 5. Assign Lane Volumes, in vph



Step 7. Sum of Critical Volumes

280 + 730 + 260 + 340

= 1610 vph

Step 8. Intersection Level of Service

(compare Step 7 with Table 6)

FAILURE * —

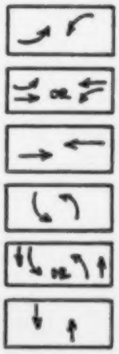
Step 9. Recalculate

Geometric Change 1 THRU LANE-APPROACHES 3 + 4
1 LT LANE-APPROACHES 1 + 2

Signal Change

Volume Change

Step 3. Identify Phasing 8φ



B2 B1

A1 B2 or A2B1

A1 A2

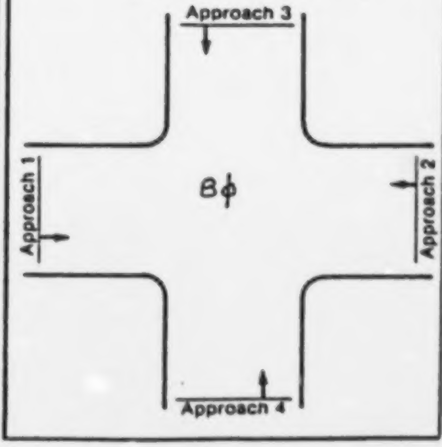
B4 B3

A3B4 or A4B3

A3 A4

A1 →	A3 ↓	B1 ↰	B3 ↱
A2 ←	A4 ↑	B2 ↗	B4 ↳

Step 6a. Critical Volumes, in vph (two phase signal)



Comments

* INTERSECTION WILL NOT OPERATE WITHOUT VERY LONG QUEUES AND EXCESSIVE DELAYS.

(Example 2)

Note: "(R)" denotes a recalculation.

Step 1(R). Identify Lane Geometry. The new lane geometry to be analyzed is shown on the form.

Step 2(R). Identify Volumes. Design hour volumes are shown on the form.

Step 3(R). Identify Phasing. An eight phase signal is assumed, with left turn arrows for each direction. Left turns are allowed only on the arrow (in a protected mode).

Step 4(R). Left Turn Check. Each left turn movement has a protected phase. Therefore, the left turn check is not needed.

Step 5(R). Assign Lane Volumes. Left turns are assigned to left turn lanes and right turns are assigned to exclusive right turn lanes, on Approaches 1 and 2. Remaining volumes are distributed equally to the remaining lanes.

Step 6(R). Critical Volumes. Using Step 3, the phase sequence which most likely will appear under volumes of Step 5 is; B2B1 , B1A2 , A1A2 , B4B3 , A3B4 , and A3A4 . For example, since the left turn volume from Approach 2 (B1) is greater than left turn volume from Approach 1 (B2), B1 will continue receiving a green arrow after B2 has been terminated. Thus, A2B1 is selected as the most probable phase, rather than A1B2.

Using the most probable phase sequence, the through plus right turn volume (except where right turns have an exclusive lane) which moves during a left arrow is subtracted from the total through plus right turn volume and the remaining volume is carried over to the next phase. Note that exclusive right turn lanes are not included in the critical volume analysis when right turns on red are permitted unless the analyst considers this lane to be critical. In this example, right turns on red are permitted.

Table 6. Level of Service Ranges

PLANNING Applications (in vph)			
Level of Service	Maximum Sum of Critical Volumes		
	Two Phase	Three Phase	Four or more Phases
A	900	855	825
B	1050	1000	965
C	1200	1140	1100
D	1350	1275	1375
[✓ E]	1500	1425	1225
F	---------not applicable---------		
OPERATIONS AND DESIGN Applications (in pch)			
(deleted)			

Step 7(R). Sum of Critical Volumes. The sum of critical volumes for all phases is 120 + 160 + 577 + 200 + 60 + 217, or 1334 vph.

Step 8(R). Intersection Level of Service. Using Table 6 1334 vph falls within the range of 1226 to 1375, for Level of Service E for eight phase control.

Step 9(R). Recalculate. Recalculations could be made to determine the improvement in level of service from other geometric or signal changes, such as addition of double left turn lanes.

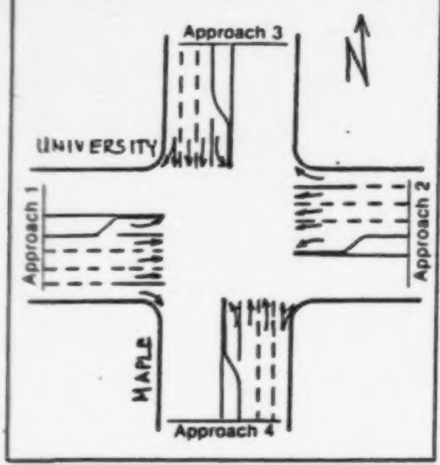
Critical Movement Analysis: PLANNING
Calculation Form 1

Exa.nple 2
(Recalculation)

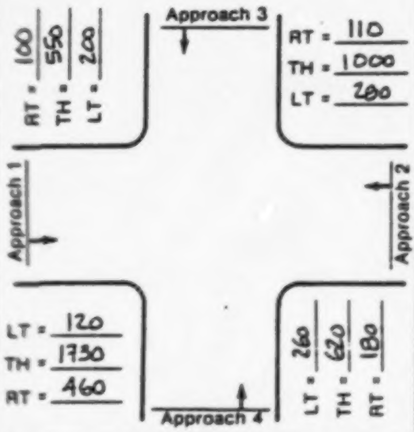
Intersection UNIVERSITY AND MAPLE **Design Hour** 4:30 - 5:30 p.m.

Problem Statement FIND CHANGE IN LOS BY ADDING ADDITIONAL THRU AND RT LANES

Step 1. Identify Lane Geometry



Step 2. Identify Volumes, in vph



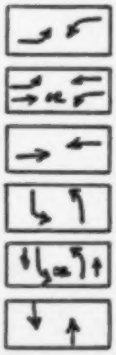
Approach 3: RT = 100 TH = 550 LT = 200

Approach 2: RT = 110 TH = 1000 LT = 280

Approach 1: LT = 120 TH = 1730 RT = 460

Approach 4: LT = 260 TH = 620 RT = 180

Step 3. Identify Phasing 8ϕ



B2 B1

A1 B2 OR A2 B1

A1 A2

B4 B3

A3 B4 OR A4 B3

A3 A4

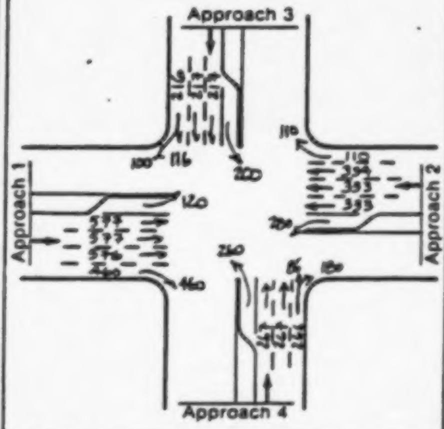
A1 → A3 ↓
A2 ← A4 ↑

B1 ↱ B3 ↰
B2 ↳ B4 ↲

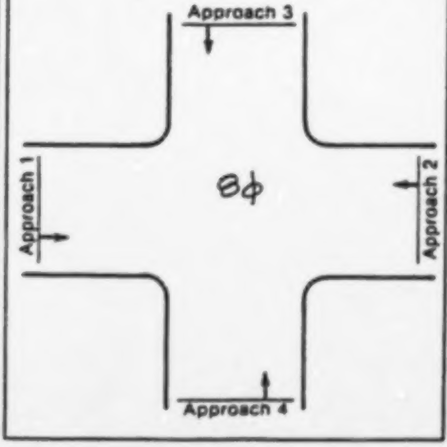
Step 4. Left Turn Check

	Approach 1	2	3	4
a. Number of change intervals per hour	8ϕ			
b. Left turn capacity on change interval, in vph				
c. G/C Ratio				
d. Opposing volume in vph				
e. Left turn capacity on green, in vph				
f. Left turn capacity in vph (b + e)				
g. Left turn volume in vph				
h. Is volume > capacity (g > f)?				

Step 5. Assign Lane Volumes, in vph



Step 6a. Critical Volumes, in vph (two phase signal)



Step 6b. Volume Adjustment for Multiphase Signal Overlap

Probable Phase	Possible Critical Volume in vph	Volume Carryover to next phase	Adjusted Critical Volume in vph
B2B1	120(B2)	280-120= 160(B1)	120
A2B1	160 (B1)	334-160=174(A2)	160
A1A2	577(A1) OR 174 (A2)		577
B4B3	200 (B4)	260-200= 60(B3)	200
A4B3	60(B3)	267-60=207(A4)	60
A3A4	217(A3) OR 207(A4)		217

Step 7. Sum of Critical Volumes

280 + 577 + 60 + 217
= 1334 vph

Step 8. Intersection Level of Service

(compare Step 7 with Table 6)

E

Step 9. Recalculate

NOT NECESSARY

Geometric Change ____

Signal Change ____

Volume Change ____

Comments

X.

PROCEDURES FOR ANALYZING TRANSPORTATION CONTROL MEASURES

SIMPLE SIGNAL TIMING CHANGES

Frequently a minor adjustment in signal timing at intersections will reduce idling emissions associated with vehicle delay. Figure 1 shows a simple intersection. The following series of assumptions are made in order to illustrate key concepts:

- The signal timing changes primarily effect peak period air pollution emissions;
- The traffic volume in the North-South direction is heavier than that in the East-West direction and the number of lanes in each direction are the same;
- The traffic volume in each direction during the peak travel period is four times the directional peak hour volume.
- The number of vehicles wishing to make left turns is almost negligible;
- This signal is at an isolated (non-coordinated) intersection and is not subject to traffic signal synchronization requirements or other special considerations (e.g., need for long intervals for pedestrians to traverse the roadway; and
- The green time to cycle time (g/c) ratios before and after adjustments are made to the signal are known. This latter information is critical since minimizing overall vehicle delay may result in excessive delay to traffic in the lower volume direction, i.e., perhaps overall vehicle delay should be subject to a constraint or maximum delay per vehicle.

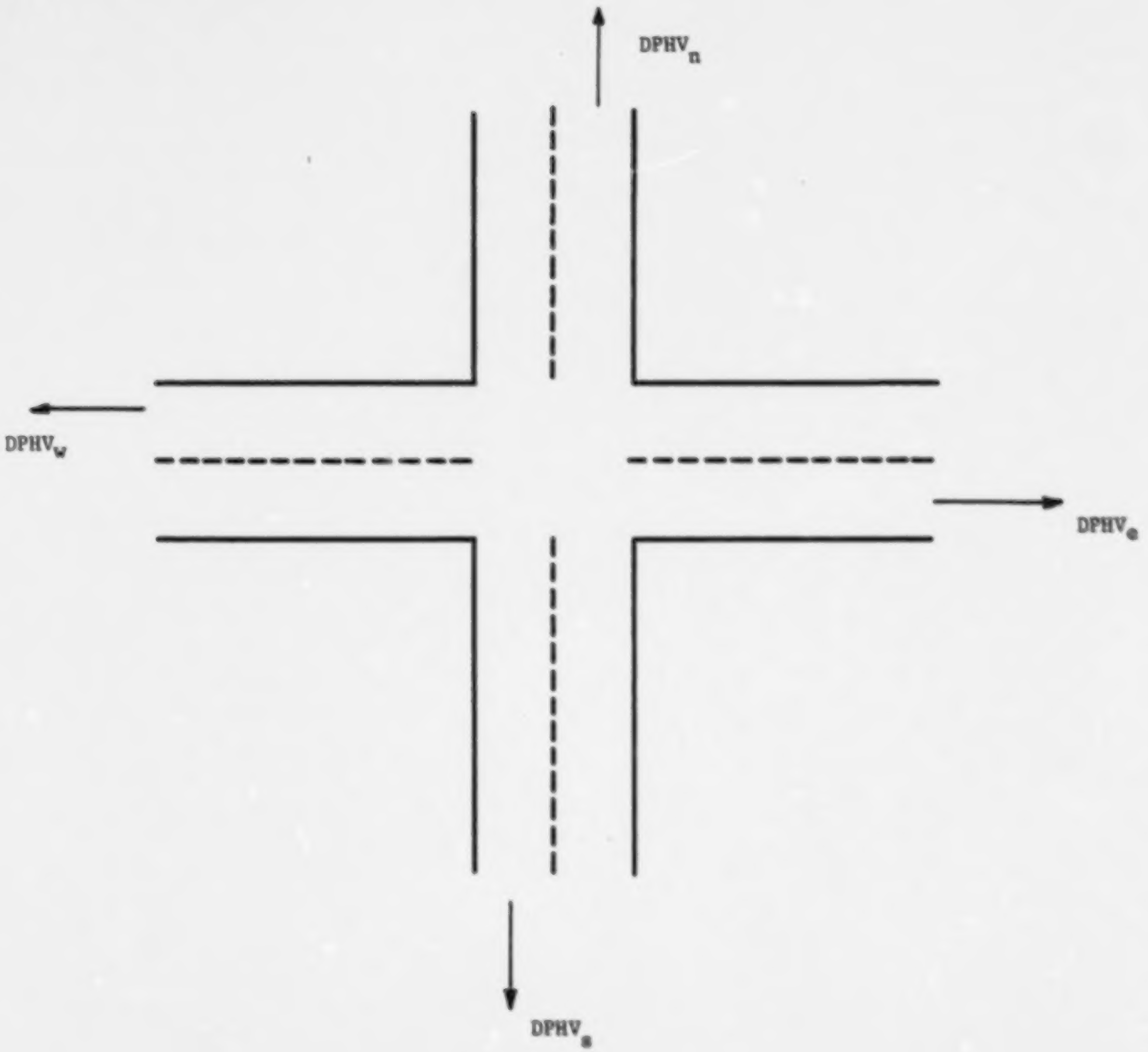
GENERAL RELATIONSHIPS

- The probability of a vehicle being delayed at an intersection, assuming uniform flow during a given travel period, may be estimated as:

$$P = \frac{(c-g)}{c}$$

FIGURE 1

SIMPLE SIGNALIZED INTERSECTION



where:

P = probability of a vehicle being delayed at the intersection

c = cycle time, hours

g = green time, hours

- The average delay per vehicle, assuming that vehicles arrive uniformly when the signal is not green, may be estimated as:

$$d = \frac{(c-g)}{2}$$

Where:

d = average delay per vehicle in hours

- The total vehicle hours of delay may be estimated as:

$$D = (\text{volume of traffic}) \times P \times d$$

Where:

D = total vehicle-hours of delay

Illustrative Application

For the simple intersection signal shown in Figure 1, the total peak travel period delay D_p may be estimated as:

$$D_p = D_{ns} + D_{ew}$$

$$D_{ns} = 4 \times (DPHV_n + DPHV_s) \times P_{ns} \times d_{ns}$$

$$D_{ew} = 4 \times (DPHV_e + DPHV_w) \times P_{ew} \times d_{ew}$$

$$D_p = 4 \times \left[(DPHV_n + DPHV_s) \times P_{ns} \times d_{ns} + (DPHV_e + DPHV_n) \times P_{ew} \times d_{ew}\right]$$

$$= 4 \times \left[(PHV_{ns}) \times P_{ns} \times d_{ns} + (PHV_{ew}) \times P_{ew} \times d_{ew}\right]$$

where:

D_p = total peak travel period vehicle-hours of delay at an intersection

D_{ns} = total vehicle-hours of delay in the North-Sourt direction during peak travel period;

D_{ew} = total vehicle-hours of delay in the East-West direction during peak travel period;

$DPHV_{n,s,w,e}$ = directional peak hour volume in the North, South, West and East directions;

P_{ns} = the probability of a vehicle being delayed in the North-South direction;

P_{ew} = the probability of a vehicle being delayed in the East-West direction;

d_{ns} = the average delay of vehicles delayed in the North-South direction; and

d_{ew} = the average delay of vehicles delayed in the East-West direction.

If a change in signal timing is made such that the green time to cycle time ratio (g/c) in the highest traffic volume direction, the North-South one, is increased, then the probability of a delay and the expected delay per delayed vehicle will also decrease in the North-South direction. The exact opposite, but to a lesser extent, will occur in the East-West direction. The net decrease in vehicle-hours of delay during the peak travel period may be estimated as follows:

$$D_p - D'_p = 4x\left[PHV_{ns}\,(P_{ns} \times d_{ns} - P'_{ns} \times d'_{ns}) + PHV_{ew}\,(P_{ew} \times d_{ew} - P'_{ew} \times d'_{ew})\right]$$

where: ' indicates after the signal has been adjusted

$PHV_{ns} = DPHV_n + DPHV_s$

$PHV_{ew} = DPHV_e + DPHV_w$

The reduction in emissions associated with the decrease in intersection idling delay during the peak period may be estimated as:

$$\text{Decrease in emissions as a result of intersection signal changes} = \frac{\text{decrease in vehicle-hours delay}}{\text{(emissions per vehicle-hour of idling)}} = \frac{D_p - D'_p}{\text{(emissions per vehicle-hour of idling)}}$$

APPENDIX A

# TRAFFIC SIGNAL WARRANTS
FROM THE
MANUAL ON UNIFORM
TRAFFIC CONTROL DEVICES

C. WARRANTS

4C-1 Advance Engineering Data Required

A comprehensive investigation of traffic conditions and physical characteristics of the location is required to determine the necessity for a signal installation and to furnish necessary data for the proper design and operation of a signal that is found to be warranted. Such data desirably should include:

1. The number of vehicles entering the intersection in each hour from each approach during 16 consecutive hours of a representative day. The 16 hours selected should contain the greatest percentage of the 24-hour traffic.
2. Vehicular volumes for each traffic movement from each approach, classified by vehicle type (heavy trucks, passenger cars and light trucks, and public-transit vehicles), during each 15-minute period of the two hours in the morning and of the two hours in the afternoon during which total traffic entering the intersection is greatest.
3. Pedestrian volume counts on each crosswalk during the same periods as the vehicular counts in paragraph (2) above and also during hours of highest pedestrian volume. Where young or elderly persons need special consideration, the pedestrians may be classified by general observation and recorded by age groups as follows:
    - (a) under 13 years
    - (b) 13 to 60 years
    - (c) over 60 years.
4. The 85-percentile speed of all vehicles on the uncontrolled approaches to the location.
5. A conditions diagram showing details of the physical layout, including such features as intersectional geometrics, channelization, grades, sight-distance restrictions, bus stops and routings, parking conditions, pavement markings, street lighting, driveways, location of nearby railroad crossings, distance to nearest signals, utility poles and fixtures, and adjacent land use.
6. A collision diagram showing accident experience by type, location, direction of movement, severity, time of day, date, and day of week for at least one year.

The following data are also desirable for a more precise understanding of the operation of the intersection and may be obtained during the periods specified in (2) above:

1. Vehicle-seconds delay determined separately for each approach.

2. The number and distribution of gaps in vehicular traffic on the major street when minor-street traffic finds it possible to use the intersection safely.

3. The 85-percentile speed of vehicles on controlled approaches at a point near to the intersection but unaffected by the control.

4. Pedestrian delay time for at least two 30-minute peak pedestrian delay periods of an average weekday or like periods of a Saturday or a Sunday.

Adequate roadway capacity at a signalized intersection is desirable. Widening of both the main highway and the intersecting roadway may be warranted to reduce the delays caused by assignment of right-of-way at intersections controlled by traffic signals. Widening of the intersecting roadway is often beneficial to operation on the main highway because it reduces the signal time that must be assigned to side-street traffic. In urban areas, the effect of widening can be achieved by elimination of parking at intersectional approaches. It is always desirable to have at least two lanes for moving traffic on each approach to a signalized intersection. Additional width may be necessary on the leaving side of the intersection, as well as the approach side, in order to clear traffic through the intersection effectively. Before an intersection is widened, the additional green time needed by pedestrians to cross the widened streets should be checked to ensure that it will not exceed the green time saved through improved vehicular flow.

4C-2 Warrants for Traffic Signal Installation

Traffic control signals should not be installed unless one or more of the signal warrants in this Manual are met. Information should be obtained by means of engineering studies and compared with the requirements set forth in the warrants. If these requirements are not met, a traffic signal should neither be put into operation nor continued in operation (if already installed).

For the purpose of warranting signalization, a wide-median intersection should be considered as one intersection.

When a traffic control signal is indicated as being warranted, it is presumed that the signal and all related traffic control devices and markings are installed according to the standards set forth in this Manual. It is further presumed that signal indications are properly phased, that roadways are properly designed, that adjacent traffic signals are properly coordinated, that there is adequate supervision of the operation and maintenance of the signal and all of its related devices, and that the traffic signal controller will be selected on the basis of engineering study and judgment.

An investigation of the need for traffic signal control should include where applicable, at least an analysis of the factors contained in the following warrants:

Warrant 1—Minimum vehicular volume.
Warrant 2—Interruption of continuous traffic.
Warrant 3—Minimum pedestrian volume.
Warrant 4—School crossings.
Warrant 5—Progressive movement.
Warrant 6—Accident experience.
Warrant 7—Systems.
Warrant 8—Combination of warrants.

4C-3 Warrant 1, Minimum Vehicular Volume

The Minimum Vehicular Volume warrant is intended for application where the volume of intersecting traffic is the principal reason for consideration of signal installation. The warrant is satisfied when, for each of any 8 hours of an average day, the traffic volumes given in the table below exist on the major street and on the higher-volume minor-street approach to the intersection. An "average" day is defined as a weekday representing traffic volumes normally and repeatedly found at the location.

MINIMUM VEHICULAR VOLUMES FOR WARRANT 1

Number of lanes for moving traffic on each approach		Vehicles per hour on major street (total of both approaches)	Vehicles per hour on higher-volume minor-street approach (one direction only)
Major Street	Minor Street		
1	1	500	150
2 or more	1	600	150
2 or more	2 or more	600	200
1	2 or more	500	200

These major-street and minor-street volumes are for the same 8 hours. During those 8 hours, the direction of higher volume on the minor street may be on one approach during some hours and on the opposite approach during other hours.

When the 85-percentile speed of major-street traffic exceeds 40 mph in either an urban or a rural area, or when the intersection lies within the built-up area of an isolated community having a population of less than 10,000, the Minimum Vehicular Volume warrant is 70 percent of the requirements above.

4C-4 Warrant 2, Interruption of Continuous Traffic

The Interruption of Continuous Traffic warrant applies to operating conditions where the traffic volume on a major street is so heavy that traffic on a minor intersecting street suffers excessive delay or hazard in entering or crossing the major street. The warrant is satisfied when, for each of any 8 hours of an average day, the traffic volumes given in the table below exist on the major street and on the higher-volume minor-street approach to the intersection, and the signal installation will not seriously disrupt progressive traffic flow.

MINIMUM VEHICULAR VOLUMES FOR WARRANT 2

Number of lanes for moving traffic on each approach		Vehicles per hour on major street (total of both approaches)	Vehicles per hour on higher-volume minor-street approach (one direction only)
Major Street	Minor Street		
1	1	750	75
2 or more	1	900	75
2 or more	2 or more	900	100
1	2 or more	750	100

These major-street and minor-street volumes are for the same 8 hours. During those 8 hours, the direction of higher volume on the minor street may be on one approach during some hours and on the opposite approach during other hours.

When the 85-percentile speed of major-street traffic exceeds 40 mph in either an urban or a rural area, or when the intersection lies within the built-up area of an isolated community having a population of less than 10,000, the Interruption of Continuous Traffic warrant is 70 percent of the requirements above.

4C-5 Warrant 3, Minimum Pedestrian Volume

The Minimum Pedestrian Volume warrant is satisfied when, for each of any 8 hours of an average day, the following traffic volumes exist:

1. On the major street, 600 or more vehicles per hour enter the intersection (total of both approaches); or where there is a raised median island 4 feet or more in width, 1,000 or more vehicles per hour (total of both approaches) enter the intersection on the major street; and
2. During the same 8 hours as in paragraph (1) there are 150 or more pedestrians per hour on the highest volume crosswalk crossing the major street.

When the 85-percentile speed of major-street traffic exceeds 40 mph in either an urban or a rural area, or when the intersection lies within the built-up area of an isolated community having a population of less than 10,000, the Minimum Pedestrian Volume warrant is 70 percent of the requirements above.

A signal installed under this warrant at an isolated intersection should be of the traffic-actuated type with push buttons for pedestrians crossing the main street. If such a signal is installed at an intersection within a signal system, it should be equipped and operated with control devices which provide proper coordination.

Signals installed according to this warrant shall be equipped with pedestrian indications conforming to requirements set forth in other sections of this Manual.

Signals may be installed at nonintersection locations (mid-block) provided the requirements of this warrant are met, and provided that the related crosswalk is not closer than 150 feet to another established crosswalk. Curbside parking should be prohibited for 100 feet in advance of and 20 feet beyond the crosswalk. Phasing, coordination, and installation must conform to standards set forth in this Manual. Special attention should be given to the signal head placement and the signs and markings used at nonintersection locations to be sure drivers are aware of this special application.

4C-6 Warrant 4, School Crossing

A traffic control signal may be warranted at an established school crossing when a traffic engineering study of the frequency and adequacy of gaps in the vehicular traffic stream as related to the number and size of groups of school children at the school crossing shows that the number of adequate gaps in the traffic stream during the period when the children are using the crossing is less than the number of minutes in the same period (sec. 7A-3).

When traffic control signals are installed entirely under this warrant:

1. Pedestrian indications shall be provided at least for each crosswalk established as a school crossing.

2. At an intersection, the signal normally should be traffic-actuated. As a minimum, it should be semi-traffic-actuated, but full actuation with detectors on all approaches may be desirable. Intersection installations that can be fitted into progressive signal systems may have pretimed control.

3. At non-intersection crossings, the signal should be pedestrian-actuated, parking and other obstructions to view should be prohibited for at least 100 feet in advance of and 20 feet beyond the crosswalk, and the installation should include suitable standard signs and pavement markings. Special police supervision and/or enforcement should be provided for a new non-intersection installation.

4C-7 Warrant 5, Progressive Movement

Progressive movement control sometimes necessitates traffic signal installations at intersections where they would not otherwise be warranted, in order to maintain proper grouping of vehicles and effectively regulate group speed. The Progressive Movement warrant is satisfied when:

1. On a one-way street or a street which has predominantly unidirectional traffic, the adjacent signals are so far apart that they do not provide the necessary degree of vehicle platooning and speed control, or

2. On a two-way street, adjacent signals do not provide the necessary degree of platooning and speed control and the proposed and adjacent signals could constitute a progressive signal system.

The installation of a signal according to this warrant should be based on the 85-percentile speed unless an engineering study indicates that another speed is more desirable.

The installation of a signal according to this warrant should not be considered where the resultant signal spacing would be less than 1000 feet.

4C-8 Warrant 6, Accident Experience

The Accident Experience warrant is satisfied when:

1. Adequate trial of less restrictive remedies with satisfactory observance and enforcement has failed to reduce the accident frequency; and

2. Five or more reported accidents, of types susceptible to correction by traffic signal control, have occurred within a 12-month period, each accident involving personal injury or property damage to an apparent extent of $100 or more; and

3. There exists a volume of vehicular and pedestrian traffic not less than 80 percent of the requirements specified either in the Minimum Vehicular Volume warrant, the Interruption of Continuous Traffic warrant, or the Minimum Pedestrian Volume warrant; and

4. The signal installation will not seriously disrupt progressive traffic flow.

Any traffic signal installed solely on the Accident Experience warrant should be semi-traffic-actuated (with control devices which provide proper coordination if installed at an intersection within a coordinated system) and normally should be fully traffic-actuated if installed at an isolated intersection.

4C-9 Warrant 7, Systems Warrant

A traffic signal installation at some intersections may be warranted to encourage concentration and organization of traffic flow networks. The Systems warrant is applicable when the common intersection of two or more major routes has a total existing, or immediately projected, entering volume of at least 800 vehicles during the peak hour of a typical weekday, or each of any five hours of a Saturday and/or Sunday.

A major route as used in the above warrant has one or more of the following characteristics:

1. It is part of the street or highway system that serves as the principal network for through traffic flow;

2. It connects areas of principal traffic generation;

3. It includes rural or suburban highways outside, entering or traversing a city;
4. It has surface street freeway or expressway ramp terminals;
5. It appears as a major route on an official plan such as a major street plan in an urban area traffic and transportation study.

4C-10 Warrant 8, Combination of Warrants

In exceptional cases, signals occasionally may be justified where no single warrant is satisfied but where two or more of Warrants 1, 2, and 3 are satisfied to the extent of 80 percent or more of the stated values.

Adequate trial of other remedial measures which cause less delay and inconvenience to traffic should precede installation of signals under this warrant.

4C-11 Factors Governing Selection of Type of Control

The principal factors that may lead to the favorable consideration of traffic-actuated control in the selection of the type of signal control include:

1. Low, fluctuating or unbalanced traffic volumes.
2. High side street traffic volumes and delays only during the peak hours.
3. The pedestrian or accident warrant is the only warrant which is met.
4. The installation is to provide for one-way movement of two-way traffic.
5. The installation is at a non-intersection location.

4C-12 Pedestrian-Actuated Control

Operation of traffic-actuated signals must take into consideration the needs of pedestrians as well as vehicular traffic. This can be accomplished in the following ways:

1. When pedestrian signals are not warranted (sec. 4D-3) in conjunction with a traffic-actuated signal installation but where occasional pedestrian movement exists and there is inadequate opportunity to cross without undue delay, pedestrian detectors shall be installed and operated as prescribed in sections 4D-6 and 7.
2. When pedestrian signals are not otherwise warranted but a pedestrian movement exists which would not have adequate crossing time during the green interval, pedestrian signals and detectors shall be installed and operated as prescribed in sections 4D-6 and 7.
3. When pedestrian signals are warranted and installed in conjunction with a traffic-actuated signal, the operation should follow the patterns described in sections 4D-6 and 7.

APPENDIX B

RAILROAD GRADE CROSSING SIGHT DISTANCE RATING*

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* This discussion is largely derived from the Railroad-Highway Grade Crossing Handbook, Technology Sharing Report, U.S. Department of Transportation, Federal Highway Administration, August 1978.

SIGHT DISTANCE

The primary requirement for the geometric design of a grade crossing is that it provide adequate sight distance for the motor vehicle operator to make an appropriate decision as to whether to stop or proceed. This sight distance will be governed largely by roadside sight obstructions, horizontal alignment and vertical alignment.

Minimum Sight Triangle

Speeds of the two vehicles (train and motor vehicle) define the distances at which a driver must be able to see a train. These distances are measured along the track and along the roadway and define a minimum sight triangle (Exhibit B-1). Distances along the highway must as a minimum be the safe stopping sight distance for a given approach speed. Distances along the track are those which would result in a train traveling a given speed arriving at the crossing at approximately the same time as the motor vehicle comes to a stop.

Exhibit B-2 shows the required distances along each traveled way used in determining the minimum sight triangle. All areas within the triangle must be clear to afford the driver adequate visibility.

Example:

Maximum train speed = 60 mph (97 kph)

Highway design speed = 60 mph (97 kph)

From Exhibit B-2, distance A along the highway in Exhibit B-1 would be 640 feet (195 m). Distance B along the track would be 704 feet (214 m). These two distances are the legs of the minimum sight triangle. A line drawn from one point to the other defines the area of the minimum sight triangle.

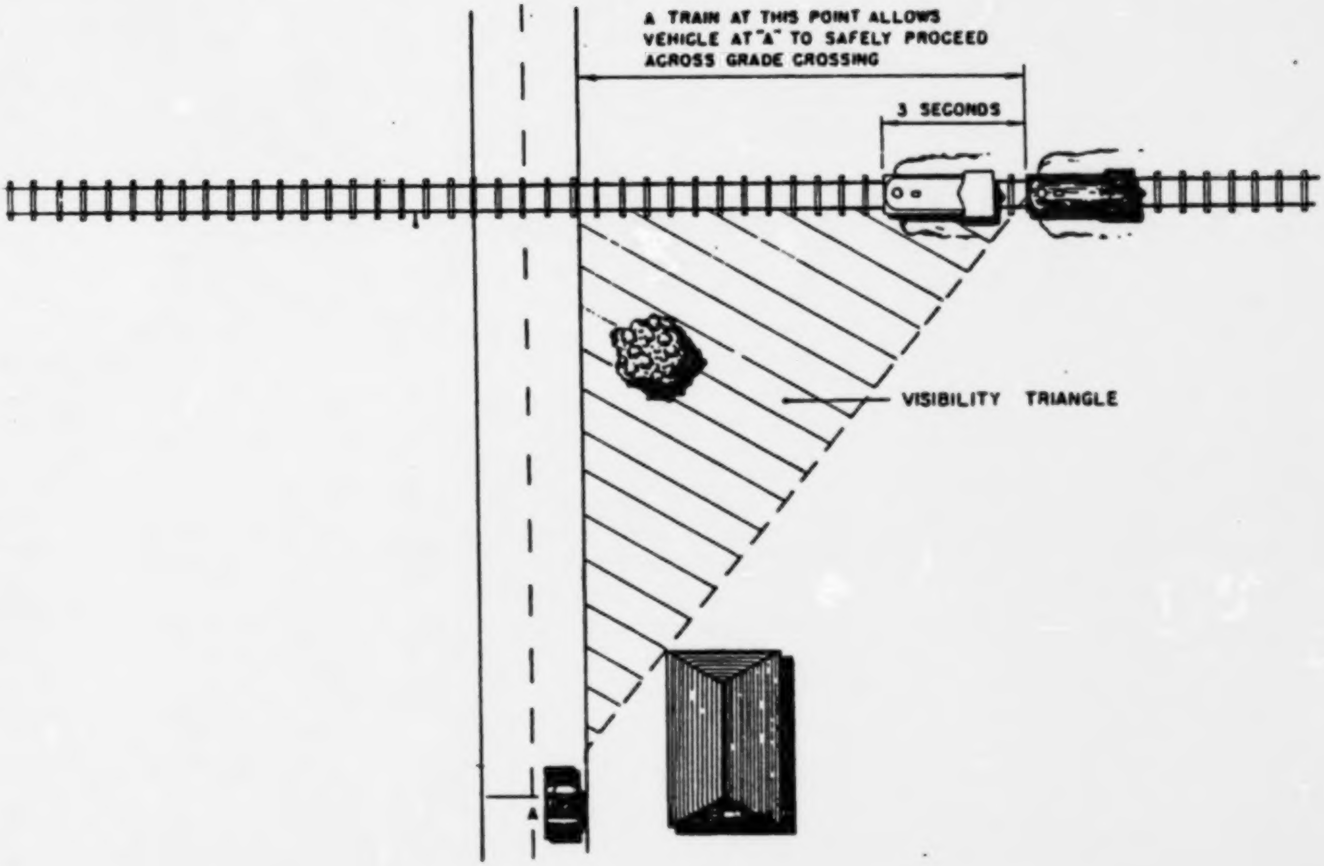
The American Association of State Highway and Transportation Officials (AASHTO) recommends a somewhat more conservative estimate of the distance measurement along the track. They suggest that for 40 mph highway traffic, the distance along the track should be at least 11 times the maximum train speed, and for 60 mph, 15 times the maximum train speed. In other words, the distance along the track for the speeds used in previous example would be:

15 x 60 mph = 900 feet

The previous calculations assumed the vehicles are approaching the crossing at highway speed. Another condition that should also receive attention is that of vehicles which are required to stop at all crossings. Required sight distances

EXHIBIT B-1

MINIMUM SIGHT TRIANGLE


A TRAIN AT THIS POINT ALLOWS VEHICLE AT "A" TO SAFELY PROCEED ACROSS GRADE CROSSING
3 SECONDS
VISIBILITY TRIANGLE
A

for stopped vehicles can be found in Table B-2 in a manner similar to that used previously. AASHTO recommends sight distances in feet not less than 13.5 times train speed in mph for single unit (SU) vehicles (buses, etc.) and 17.5 times the train speed for semi-trailer vehicles. Special attention should be given to the case of stopped vehicles to ensure that adequate sight distance is available as stopped vehicles often include buses which represent a high "risk" factor and reduced operational capabilities.

In the event that it is impossible to achieve the minimum sight triangle, careful consideration should be given to the installation of active control devices.

Obstructions

In many cases, restricted sight distance will be the result of obstructions in the sight triangle. If such obstructions are due to vegetation or other natural features, they should be removed. Other obstructions such as buildings may be difficult to remove. If such is the case, two other alternatives are available. First, advisory speed plates mounted with the advance warning sign may be used to reduce motor vehicle speeds to a level at which the minimum sight triangle is available. Such advisory speed should not be less than 20 mph (30 kph).[1/] The second alternative, as discussed previously, is the installation of active control devices.

Depending on horizontal and vertical alignment, there may not be sufficient sight distance along the roadway for motorists to properly respond to active devices. Although such cases may be rare, proper treatment is important. Where conditions are such that neither the minimum sight triangle nor stopping sight distance to active devices is attainable, flashing yellow beacons may be added to the advance warning sign. These beacons should be connected to the active devices so that when a train is approaching the obscured crossing, the beacons flash, giving the motorist adequate time to respond. If flashing beacons are added to the standard advance warning sign, a supplementary plate bearing the legend "BE PREPARED TO STOP WHEN FLASHING" should be added.

[1/] 1 mph = 1.61 kph
1 foot = .304 meters

EXHIBIT B-2

REQUIRED DESIGN SIGHT DISTANCES FOR COMBINATIONS OF HIGHWAY AND TRAIN VEHICLE SPEEDS

Train Speed	Highway Speed in MPH							
	0	10	20	30	40	50	60	70
	Distance Along Railroad From Crossing							
10	162	126	94	94	99	107	118	129
20	323	252	188	188	197	214	235	258
30	484	378	281	281	295	321	352	387
40	645	504	376	376	394	428	470	516
50	807	630	470	470	492	534	586	644
60	967	756	562	562	590	642	704	774
70	1129	882	656	656	684	750	822	904
80	1290	1008	752	752	788	856	940	1032
90	1450	1134	844	844	884	964	1056	1160
	Distance Along Highway From Crossing							
	20	65	125	215	330	470	640	840

NOTE: 1 mph = 1.61 kph
1 foot = .304 metres

SOURCE: Hedley, W. J., State of the Art Report on Railroad-Highway Grade Surfaces, U.S. Department of Transportation, Federal Highway Administration, 1973